PROCEEDINGS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS

(INSTITUTED 1852.)

VOL. XXVI.

JANUARY TO DECEMBER, 1900.

NEW YORK:

PUBLISHED BY THE SOCIETY.

1900.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXVI. JANUARY, 1900. No. 1.

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OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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PAPERS AND DISCUSSIONS.

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The prices of publications are as follows: *Proceedings*, $6 per annum; *Transactions*, $10 per annum. Postage will be added when *Proceedings* are sent to foreign countries.

American Society of Civil Engineers.

OFFICERS FOR 1900.

President, JOHN FINDLEY WALLACE.

Vice-Presidents.

Term expires January, 1901:	*Term expires January, 1902:*
ROBERT CARTWRIGHT.	RUDOLPH HERING,
ROBERT MOORE.	ALFRED NOBLE.

Secretary, CHARLES WARREN HUNT.

Treasurer, JOSEPH M. KNAP.

Directors.

Term expires January, 1901:	*Term expires January, 1902:*	*Term expires January, 1903:*
S. L. F. DEYO,	JOHN A. BENSEL,	JOHN F. O'ROURKE,
JOHN KENNEDY,	C. W. BUCHHOLZ,	HENRY B. SEAMAN,
HENRY MANLEY,	SAMUEL WHINERY,	THOMAS H. JOHNSON,
CHARLES C. SCHNEIDER,	EDMUND K. TURNER,	JOSEPH RAMSEY, JR.,
JOHN J. McVEAN,	PALMER C. RICKETTS,	HENRY B. RICHARDSON,
GEORGE Y. WISNER.	JAMES D. SCHUYLER.	GEORGE A. QUINLAN.

Assistant Secretary, T. J. McMINN.

Standing Committees.

THE PRESIDENT OF THE SOCIETY IS *ex-officio* MEMBER OF ALL COMMITTEES.

On Finance:	*On Publications:*	*On Library:*
SAMUEL WHINERY,	RUDOLPH HERING,	JOHN A. BENSEL.
S. L. F. DEYO,	PALMER C. RICKETTS,	C. W. BUCHHOLZ,
HENRY MANLEY,	JOHN F. O'ROURKE,	JOHN KENNEDY,
ROBERT CARTWRIGHT.	JAMES D. SCHUYLER,	ROBERT MOORE,
HENRY B. SEAMAN.	ALFRED NOBLE.	CHARLES WARREN HUNT.

Special Committees.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 A.M. to 10 P.M.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - - - 533 Columbus.
CABLE ADDRESS, - - - - - "Ceas, New York."

Vol. XXVI. JANUARY, 1900. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

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SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

January 3d, 1900.—The meeting was called to order at 8.30 P. M., Vice-President Edward P. North in the chair; Charles Warren Hunt, Secretary; and present, also, 95 members and 28 guests.

The minutes of the meetings of December 6th and 20th, 1899, were approved as printed in *Proceedings* for December, 1899.

A paper by Allen Hazen, Assoc. M. Am. Soc. C. E., entitled, "The Albany Water Filtration Plant," was presented by the author and illustrated with stereopticon views. The paper was discussed by Messrs. George I. Bailey, William B. Fuller, P. A. Maignen, George Hill, J. F. O'Rourke, Rudolph Hering, A. M. Miller, William P. Mason and the author. The Secretary read discussions on the subject by Messrs. C. E. Fowler and George W. Fuller.

Ballots were canvassed, and the following candidates declared elected:

As Members.

John Abiel Atwood, Beaver, Pa.
James Hayward Bacon, St. Augustine, Fla.
Eduardo Justo Chibas, Guantanamo, Cuba.
Arthur Graham Glasgow, London, Eng.
Charles Addison Hook, Baltimore, Md.
Otis Ellis Hovey, Athens, Pa.
Milton Gerry Starrett, New York City.
Enrique Augusto Touceda, Albany, N. Y.

As Associate Members.

William Frederick Allen, New York City.
Edmund Canby Alsop, Philadelphia, Pa.
Frank Raymond Coates, Stamford, Conn.
Frank Grant Cudworth, Brooklyn, N. Y.
John Maurice Evans, Albany, N. Y.
Joseph Kendall Freitag, Boston, Mass.
Gisli Gudmundsson, Pittsburg, Pa.
Arthur Stanley Ives, New York City.
Ernst Fredrik Jonson, New York City.
John Bigger Leeper, Pittsburg, Pa.
William Griffith Moler, Springfield, Ohio.
Percival Mosley Sax, Philadelphia, Pa.
Frank Stone Tainter, Far Hills, N. J.
Fred Conover Warman, Washington, D. C.
Charles William Schrage Wilson, New Rochelle, N. Y.

Announcement was made that the following candidates were elected by the Board of Direction, January 2d, 1900:

As Juniors.

Arthur Horace Blanchard, Providence, R. I.
William McKeever, New York City.
Jerome Frederick Wilhelm, Grand Rapids, Mich.
Percy Hartshorne Wilson, Camden, N. J.
Moses Hannibal Wright, Nashville, Tenn.

The Secretary announced that at the meeting of the Board of Direction, January 2d, 1900, the ballot on the reconsideration of John Knickerbacker was canvassed, and that Mr. Knickerbacker was declared elected as a Member of the Society.

The Secretary read the programme for the Forty-seventh Annual Meeting.

Adjourned.

FORTY-SEVENTH ANNUAL MEETING.*

January 17th, 1900.—The meeting was called to order at 10.15 A.M., President Desmond FitzGerald in the chair; Charles Warren Hunt, Secretary; and present, also, 295 members and 29 guests.

Messrs. T. McC. Leutzé and H. M. Rood were appointed tellers to canvass the ballots for officers for the ensuing year.

On motion, the reading of the minutes of the meeting of January 3d, 1900, was dispensed with.

The Annual Report† of the Board of Direction for the year ending December 31st, 1899, and the Annual Reports of the Treasurer and the Secretary were presented, and, on motion, duly seconded, accepted.

The Secretary read the report of the Committee to recommend the Award of Prizes, and reported that the action of the Board of Direction in regard thereto was as follows:

That the Collingwood Prize be awarded to Julius Kahn, Jun. Am. Soc. C. E., for Paper No. 846, entitled "The Coal Hoists of the Calumet and Hecla Mining Company"; that the Thomas Fitch Rowland Prize be awarded to R. S. Buck, M. Am. Soc. C. E., for Paper No. 836, entitled "The Niagara Railway Arch," and that the Norman Medal be awarded to E. Herbert Stone, M. Am. Soc. C. E., for Paper No. 850, entitled "The Determination of the Safe Working Stress for Railway Bridges of Wrought Iron and Steel."

The Secretary presented the report of the Board of Direction in relation to the expenses of the Nominating Committee, and, in relation to this subject, read a letter from J. A. Ockerson, M. Am. Soc. C. E., and a proposed amendment to the Constitution, signed by five Corporate Members.‡

The proposed amendments were discussed, informally.

The Secretary presented a report of a committee, appointed by the Board of Direction, to report on the acoustics of the Auditorium.

In relation to this subject, Rudolph Hering, M. Am. Soc. C. E., Chairman of the Committee, addressed the meeting, and the President stated that the Board had already taken action in regard to carrying out the recommendations of the committee.

The Secretary presented a report§ from Sandford Fleming, M. Am. Soc. C. E., Chairman of the Committee on Standard Time.

After discussion the following resolution was offered by Mendes Cohen, Past-President Am. Soc. C. E.:

"*Resolved:* That the Final Report of the Committee on Standard

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* A full report of the Forty-seventh Annual Meeting will be published in the February number of *Proceedings*.

† See pages 7 to 16 for the Annual Reports of the Board of Direction, the Treasurer and the Secretary.

‡ This report, and the proposed amendments, will be printed in full in the February number of *Proceedings*.

§ This report will be printed in full in the February number of *Proceedings*.

Time be accepted, and that the thanks of the Society be conveyed to Mr. Fleming and the members of the Committee for their long-continued service."

The resolution was adopted.

George F. Swain, M. Am. Soc. C. E., Chairman of the Committee on the Proper Manipulation of Tests of Cement, presented the report of that committee.*

On motion by Edward P. North, Vice-President, Am. Soc. C. E., it was ordered that the report be accepted and placed on file, and that the Committee be continued.

The Secretary read a letter from Mr. Louis A. Risse, Chief Engineer of the Board of Public Improvements of the City of New York, inviting the members of the Society to inspect a large topographical map of the City of New York, prepared by the Topographical Bureau of the Board of Public Improvements, for the International Exhibition in Paris.

A. McC. Parker, M. Am. Soc. C. E., presented an invitation to the members of the Society to inspect the working of a rubber belt conveyor now being used in handling material which is being excavated from a large foundation at 38th St. and First Ave., New York City.

The Secretary made announcements in reference to the Annual Convention of 1900, and Elmer L. Corthell, M. Am. Soc. C. E., addressed the Society on that subject and on the Engineering and Navigation Congresses to be held in Paris during the Exposition.

The Secretary presented the report† of the Tellers appointed to count the votes for officers for the ensuing year.

The President announced that the following officers were elected for the year 1900:

President, to serve one year:

John Findley Wallace, Chicago, Ill.

Vice-Presidents, to serve two years:

Rudolph Hering, New York City.

Alfred Noble, Chicago, Ill.

Treasurer, to serve one year:

Joseph M. Knap, New York City.

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* This report will be printed in full in the February number of *Proceedings*.

† On account of the necessity for going to press at an early hour this report and the several others presented, could not be printed in this number of *Proceedings*. They, together with the minutes of the meeting in full, will be printed in the February *Proceedings*.

Directors, to serve three years:

District No. 1.—JOHN F. O'ROURKE, New York City.
District No. 1.—HENRY B. SEAMAN, New York City.
District No. 4.—THOMAS H. JOHNSON, Pittsburg, Pa.
District No. 6.—JOSEPH RAMSEY, Jr., St. Louis, Mo.
District No. 6.—HENRY B. RICHARDSON, New Orleans, La.
District No. 7.—GEORGE A. QUINLAN, Houston, Tex.

Mr. Wallace, President for 1900, was introduced to the Society by Mr. FitzGerald and took the chair.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

January 2d, 1900.—Vice-President North in the chair; Chas. Warren Hunt, Secretary; and present, also, Messrs. Bensel, Buchholz, Haines, Hering, Owen, Ricketts, Schneider, Thomson, Turner and Whinery.

Action was taken in regard to members in arrears for dues.

The Report of the Board of Direction to the Society for the year 1899 was adopted.

Ballots were canvassed and THOMAS FITCH ROWLAND, M. Am. Soc. C. E., and GEORGE WALLACE MELVILLE were declared elected Honorary Members of the Society, the election to date from the receipt of the last ballot, December 20th, 1899.

Ballots were canvassed on the reconsideration of the ballot in the case of John Knickerbacker, and Mr. Knickerbacker was declared elected a Member of the Society.

The following resignations were accepted, taking effect December 31st, 1899: Charles Jarvis Bates, M. Am. Soc. C. E.; Richard Despard Dodge, M. Am. Soc. C. E.; Richard Morley Harison, M. Am. Soc. C. E.; Arthur Hillman Scott, M. Am. Soc. C. E.; Thomas Osborn Horton, Jun. Am. Soc. C. E.; Henry Hollister Robinson, Jun. Am. Soc. C. E.

Applications were considered and other routine business transacted.

Five candidates for Junior were elected.

The meeting adjourned, subject to the call of the Chair.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 A. M. to 10 P. M.

MEETINGS.

Wednesday, February 7th, 1900, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by William B. Landreth, M. Am. Soc. C. E., entitled "The Improvement of a Portion of the Jordan Level of the Erie Canal," will be presented. This paper is printed in the *Proceedings* for December, 1899.

Wednesday, February 21st, 1900, at 8.30 P. M., a regular meeting will be held, at which a paper by Charles S. Gowen, M. Am. Soc. C. E., entitled "The Foundations of the New Croton Dam," will be presented. This paper is printed in this number of *Proceedings*.

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31ST, 1899.

PRESENTED AT THE ANNUAL MEETING, JANUARY 17TH, 1900.

The Board of Direction, in compliance with the Constitution of the Society, presents its report for the year ending December 31st, 1899.

MEMBERSHIP.

The changes in membership are shown in the following table:

	Jan. 1st, 1899.			Jan. 1st, 1900.			Losses.				Additions.		Totals	
	Resident.	Non-Resident.	Total.	Resident.	Non-Resident.	Total.	Transfer.	Resignation.	Dropped.	Death.	Transfer.	Election.	Loss.	Gain.
Honorary Members......	2	7	9	1	8	9	..	..	....	3	*2	1	3	3
Corresponding Members	....	3	3	....	3	3	..	..	....	....	....	....	....	...
Members................	253	1 078	1 331	257	1 118	1 370	2	7	9	17	†19	55	35	74
Associate Members......	75	295	370	117	307	424	16	..	9	2	‡28	53	27	81
Associates..............	35	51	86	31	62	93	..	..	1	1	‡2	7	2	9
Juniors..................	95	164	259	107	156	262	33	6	6	....	....	48	45	48
Fellows..................	10	28	38	12	26	38	..	..	....	1	....	1	1	1
Subscribers..............	3	25	28	3	25	28	..	..	....	....	....	....	....	...
Totals..............	473	1 651	2 124	528	1 700	2 227	51	13	25	24	51	165	113	216

* Members. † 16 Associate Members and 3 Juniors. ‡ Juniors.

It will be seen by the table that the net increase during the year was 103.

The total number of applications received during the year was 259.

Action by the Board has been taken as follows:

Passed to ballot as Members......................	87
Passed to ballot as Associate Members............	102
Elected Associates...............................	8
Elected Juniors..................................	47
Elected Fellow...................................	1
Total................................	245
Applications now awaiting action.....	21

The losses by death during the year number 24. They are as follows:

Three Honorary Members: Julius Walker Adams, George Sears Greene, Horatio Gouverneur Wright.

Seventeen Members: Winthrop Bartlett, Jacob Blickensderfer, Charles Irwin Brown, Clarence Allan Carpenter, James Duane, Robert Gillham, Horace Harding, Alfred Ephraim Hunt, Archibald Johnson, Samuel Killebrew, John Charles O'Melveny, Francis Rinecker, Robert Delos Rowe, Karl Spörck, Charles Herbert Swan, Frederic Candee Weir, Arthur Sheppard Campbell Würtele.

Two Associate Members: William Roberts Michie, Albert Howell Porter.

One Associate: Herbert Steward.

One Fellow: Charles Carroll Gilman.

Library.

The following sums have been expended during the year:

For Binding 23 Volumes	$47.10
" Purchase of Guide-Books, Directories, etc.	26.25
	$73.35

The additions to the Library from all sources during the year were:

Bound Volumes	268
Unbound Volumes	118
Pamphlets	257
Maps, Photographs, Charts	36
Specifications	297
Total additions	976

The total attendance registered in the Reading Room during the year was 1 761.

In its last report the Board referred to the work of reclassifying and indexing the Library as being well under way. It now has the pleasure to report that this work is almost finished. The Library has been divided into general classes, each of which is subdivided as necessity, in each case, seemed to require, the governing idea being to so arrange the books, and to so group the cards, as to enable one unaccustomed to the technicalities of library work to investigate a general

subject or pursue a specific inquiry with the least possible loss of time, and by a personal use of the index.

The General Classes are as follows, the number of titles indexed under each being indicated by figures. The work has not been finished in classes marked thus.*

Railroads	3 042	Military	221
Railroads, Street	220	Mining	148
Water-ways	2 425	Roads and Pavements	76
Water Supply	2 194	*Municipal	—
Sanitation	754	Landscape Architecture	40
Bridges	571	Geography	141
Mechanical	244	*Engineering Hand Books	—
Electric	207	*Society Publications	3
Gas	27	*Periodicals	—
Architecture and Building	304	Dictionaries and Encyclopedias	40
Marine	251	*General Science	573
*Miscellaneous	142		

The total number of titles classified to date is 11 623, representing 18 055 volumes, pamphlets, maps, photographs and specifications, which have been accessioned, catalogued, labeled and arranged on the shelves.

The estimated number of titles awaiting classification is 1 500.

The plan of indexing provides also for cross-references between classes, for a separate "Author" catalogue, and for a "Subject" catalogue.

Under the latter are brought together for ready reference analytical details, the discovery of which would otherwise require much search in the books themselves.

The total number of cards written to date is 42 057.

The Board has already had under consideration the advisability of publishing, as soon as the index is complete, a catalogue of the library for distribution to the membership, and it is believed that the financial status of the Society will warrant this being done, and that when it is accomplished the result of the expenditure of time and money on the Library during the past two years, which cannot have been apparent to members, will be satisfactory to them, and particularly to those who are non-resident, and have had heretofore no means of knowing either the number or character of the books upon our shelves.

In April, 1899, arrangements were made with some prominent publishers of engineering books, to contribute the works issued by them promptly to the Library, and since that date, under the heading "New

Books of the Month," brief book notices have been published in *Proceedings*. The net result of this has been the receipt of 91 bound volumes, representing a value of $194.50, which otherwise it would have been necessary to purchase, and some of which would probably never have been received. The publication of these notices, moreover, it is believed, is of considerable advantage and convenience to engineers.

PUBLICATIONS.

At its March meeting, the Board decided to publish in *Proceedings*, as well as in *Transactions*, all discussion on papers presented, the objects being to secure cross-discussion and to bring the current topics promptly before all members. The interest which this system has provoked may be seen clearly in the character and number of communications, published since that time, on the various subjects under discussion, and it is now evident from the results that the *Proceedings* have a current value which was heretofore lacking.

In the March Number of *Proceedings* there was begun, in an experimental way, a "Monthly List of Recent Engineering Articles of Interest," which has been continued in each subsequent issue. This list has grown rapidly. In the March Number references were given to 59 articles from 19 publications, the list being unclassified; in the April Number the list covered 47 publications, and it became necessary to classify the entries; and in the December Number the list of periodicals referred to was 63, and 305 articles were listed. The total number of titles of articles classified and printed in eight Numbers of *Proceedings* has been 2 229.

The following tables give in detail a summary of the publications issued during the year.

	Number issued.	Total edition of each Number.	Number of Pages.	Plates.	Cuts.
*Transactions**	2	2 500	1 251	12	166
*Proceedings**	10	Average 2 600	1 335	43	204
Constitution and List of Members	1	2 700	205	..	....
Advertisements	10	Average 2 600	150	..	....
Totals	..	......	2 941	55	370

* Includes Indexes and Tables of Contents.

The cost of publications has been :

For Paper, Printing, Binding, etc., *Transactions* and *Proceedings*		$7 951.02
For Plates and Cuts		1 173.90
For Boxes, Mailing Lists, Copyright and sundry expenses.		203.41
For Commission on Advertisements		256.45
For 9 800 extra copies of Papers and Memoirs		534.10
For List of Members		861.02
Total		$10 979.90
For time of officers and clerks charged to publications		3 495.87
Total		$14 475.77
Deduct amount received for advertisements	$2 310.70	
Deduct amount received for sale of publications	2 365.00	
		4 675.70
Net cost of publications		$9 800.07
Net cost of publications for 1898 (see Report of Board of Direction, January, 1899)		9 958.55

MEETINGS.

There have been 32 meetings held during the year, as follows: At the Annual Meeting, 2; at the Annual Convention, 5; regular semi-monthly meetings, 18; special meeting, 1; Junior meetings, 6.

At these meetings 18 formal papers were presented, 5 illustrated lectures were given, and 13 topics were informally discussed.

The attendance at the Forty-sixth Annual Meeting was 316 members and many guests, and at the Thirty-first Annual Convention, held at Cape May, N. J., the total attendance was 635, the largest meeting in the history of the Society.

MEDALS AND PRIZES.

The Norman Medal, for the year ending with the month of July, 1898, was awarded to B. F. Thomas, M. Am. Soc. C. E., for his paper on "Movable Dams."

The Thomas Fitch Rowland Prize, for the year ending with the month of July, 1898, was awarded to Henry Goldmark, M. Am. Soc. C. E., for his paper, entitled "The Power Plant, Pipe Line and Dam of the Pioneer Electric Power Company at Ogden, Utah."

No award of The Collingwood Prize for Juniors was made.

Finances.

The reports of the Secretary and of the Treasurer, appended to this report, give in detail the receipts and expenditures for the year, and the balances on hand at the beginning and end of the year. They show that the financial affairs of the Society are in a very satisfactory condition.

From the balance on hand the Board has decided to make a payment of $10 000 on the debt incurred for the New House of the Society, thus reducing that debt to $75 000. The Board considers it desirable to pay off this debt as rapidly as possible without in any way restricting expenditures necessary to maintain and promote the usefulness of the Society, and to that end has adopted a resolution recommending that in future an annual payment be made upon the debt, the amount of which payment, in any one year, shall be not less than the sum received for entrance fees in that year. If this recommendation is followed, it will practically result in setting apart the entrance fees of new members as a fund to be applied to the extinguishment of this debt, leaving the income from annual dues to be applied to the payment of current expenses and to enlarging the work of the Society in such ways as experience may develop.

Judging from results during recent years, the amount received from entrance fees is not likely to fall below $3 500, and will probably average at least $3 750 per year. Assuming that the debt may be reduced by the payment of the last-named sum annually, the whole principal of the debt will be extinguished in twenty years.

The Board had hoped to be able to present with this report a balance sheet showing accurately the condition of the finances of the Society and its several accounts, but its preparation has been delayed pending a proper valuation of some of the assets of the Society. It is believed that before the next annual report such a balance sheet will be ready for presentation to the Society.

By order of the Board of Direction.

CHAS. WARREN HUNT,

Secretary.

REPORT OF THE TREASURER.

In compliance with the provision of the Constitution, the Treasurer presents the following report for the year ending December 31st, 1899:

Balance on hand December 31st, 1898			$7 699.91
Receipts from current sources, January 1st to December 31st, 1899			47 215.22
Received from subscriptions to New Society House			225.00
Received from sale of Historical Sketch			80.00
Payment on audited vouchers for current business, January 1st to December 31st, 1899		$41 491.77	
Balance on hand December 31st, 1899:			
In Union Trust Company	$5 252.63		
In Garfield National Bank	7 490.73		
In hands of the Treasurer	985.00	13 728.36	
		$55 220.13	$55 220.13

Respectfully submitted,

JOHN THOMSON,

Treasurer, Am. Soc. C. E.

NEW YORK, January 8th, 1900.

REPORT OF THE SECRETARY FOR THE

To the Board of Direction of the

Gentlemen,—I have the honor to present a balance sheet of Re-
December 31st, 1899, to which is appended a table showing the com-
during the year, and its distribution to the several accounts.

New York, January 8th, 1900.

Receipts.

Balance on hand December 31st, 1898, in Bank and Trust Company and in the hands of the Treasurer		$7 699.91
Entrance Fees	$ 4 075.00	
Current Dues	24 508.32	
Past Dues	1 812.79	
Advance Dues	8 379.78	
Certificates of Membership	188.00	
Badges	655.00	
Sales of Publications	2 310.70	
Advertisements	2 365.00	
Interest	144.07	
Library	13.00	
Compounding Dues	440.00	
Fellowship Fund	250.00	
Miscellaneous	92.24	
New Society House (Subscriptions)	225.00	
Annual Meeting	839.00	
Sale of Historical Sketch	80.00	
Binding	1 142.32	
		47 520.22
		$55 220.13

YEAR ENDING DECEMBER 31st, 1899.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

ceipts and Disbursements for the fiscal year of the Society, ending
pensation paid to persons who have been in the service of the Society

Respectfully submitted,

CHAS. WARREN HUNT,
Secretary.

DISBURSEMENTS.

General Printing and Stationery	$ 1 468.78	
Publications	14 219.32	
Commission on Advertisements	256.45	
Postage	2 257.00	
Library	3 804.64	
Janitor	948.62	
Badges	494.55	
Contingencies	421.61	
Gas and Water	686.99	
Finance and Accounts	1 665.00	
House Supplies—Furniture	273.52	
Certificates of Membership	116.26	
Fuel	283.76	
Convention and Annual Meeting	1 997.13	
Insurance—Safe Deposit	6.00	
Norman Medal, Rowland and Collingwood Prizes	156.82	
Interest and Taxes	4 581.25	
Current Business	6 902.71	
Repairs and Betterments	78.20	
Binding	868.45	
Work of Committees	4.71	
		41 491.77
Balance on hand December 31, 1889		
In Garfield National Bank	$5 252.63	
In Union Trust Company	7 490.73	
In Hands of Treasurer	985.00	
		13 728.36
		$55 220.13

TABLE SHOWING THE COMPENSATION PAID TO PERSONS WHO HAVE BEEN IN THE SERVICE OF THE SOCIETY DURING THE YEAR, AND ITS DISTRIBUTION TO THE SEVERAL ACCOUNTS ACCOMPANYING REPORT OF THE SECRETARY:

NAME.	Publications.	Current Business.	Finance and Accounts.	Library.	Janitor.	Total.
Chas. Warren Hunt, Secretary	$1 775.00	$2 876.98	$555.00	$735.00		$5 941.98
John Thomson, Treasurer			100.00			100.00
T. J. McMinn, Asst. Secretary	1 200.00	600.00				1 800.00
B. J. Burke, Clerk		1 000.00				1 000.00
D. J. Mullen, Stenographer		900.00				900.00
M. F. Huckell, Bookkeeper			800 00			800.00
E. H. Frick, Asst. Librarian				900.00		900.00
E. A. Angell, Asst. Librarian				600.00		600.00
William Waldele, Office Boy		22.00				22.00
Louis Gloor, Hall Boy		186.00				186.00
Percy Harrold, Hall Boy		156.00				156.00
Frank E. Harrold, Janitor					$600.00	600.00
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MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(December 11th, 1899, to January 9th, 1900.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London). International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c
(14) *The Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor. 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*. Stockholm, Sweden.
(57) *Teknisk Ugeblad*. Christiania, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(60) *Bridges and Framed Structures*, 358 Dearborn St., Chicago, Ill., 30c.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

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The Scientific Principles of Public Lighting by Arc Lamps. F. W. Carter. (26) Serial beginning Dec. 22, ending Dec. 29.
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Le Phare de Raz-Tina (Tunisie). (35) Dec. 4.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

MATERIALS OF ENGINEERING

Part I. A Treatise on Non-Metallic Materials of Engineering; Stone, Timber, Fuel, Lubricants, etc. Part II. A Treatise on Iron and Steel. By Robert H. Thurston, M. Am. Soc. C. E. Part I, Fifth Revised Edition. Part II, Seventh Revised Edition. Cloth, 9 x 6 ins., illus. New York, John Wiley & Sons, 1898–99. Part I, $2.00; Part II, $3.50.

The subjects covered by these volumes are: Part I.—Stones and Cements; Strength of Timber, Adaptations, Preservations; Fuels; Lubricants; Leather, Belting, Paper, Rubber, Cordage. Part II.—Qualities of the Metals; History, Principles, Materials of Metallurgical Work; Historical Sketch of Iron Manufacture; The Ores of Iron; Reduction of Ores, Production of Cast Iron; Manufacture of Wrought Iron; Manufacture of Steel; Chemical and Physical Properties of Iron and Steel; Strength of Iron and Steel; Effect of Temperature and Time on Resistance; Specifications, Tests, Inspection.

Part III, which treats of The Alloys and Their Constituents, is already in the Library.

RULES FOR RAILWAY LOCATION AND CONSTRUCTION

Of The Northern Pacific Ry. Co. By E. H. McHenry, M. Am. Soc. C. E. (Published by Permission.) Cloth, 7 x 4 ins., 74 pp., 3 plates. New York, Engineering News Publishing Co., 1899. 50 cents.

The Contents are: Construction and Engineering Departments; Location; Surveys and Construction; Track and Ballast; Bridges and Culverts; Accounting and Miscellaneous; Supplies.

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ADDITIONS.

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Pres. The Continental Iron Works, Brooklyn, N. Y., Res., 329 Madison Ave., New York City .... M. Sept. 1, 1886; Hon. M. Dec. 20, 1899

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14 Daily Record Bldg., Baltimore, Md .... Jan. 3, 1900

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Eng. in charge of Office of Union Bridge Co., Athens, Pa .... Assoc. M. April 4, 1894; M. Jan. 3, 1900

Knickerbacker, John
Prest. & Eng., Eddy Valve Co. of Waterford, N. Y. Res., 86 First St., Troy, N. Y .... Jan. 2, 1900

Lippincott, Joseph Barlow
406 Byrne Bldg , Los Angeles, Cal .... Dec. 6, 1899

Palmer, Frederick
Dehri Bridge, Bengal, India .... Oct. 4, 1899

Starrett, Milton Gerry
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Flinn, Alfred Douglas
3 Mt. Vernon St., Boston, Mass. .... Dec. 6, 1899

Freitag, Joseph Kendall
166 Devonshire St., Boston, Mass .... Jan. 3, 1900

Gudmundsson, Gisli
Room 317, Telephone Bldg., Pittsburg, Pa. .... Jan. 3, 1900

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RICHARDSON, JOHN FRANCIS Asst. City Engineer, New Orleans, La.	Oct. 3, 1899
TOWNE, JOSEPH MINOTT 54 Walnut St., East Orange, N. J.	Dec. 5, 1899
WILHELM, JEROME FREDRICK Engineering Corps, G. R. & I. Ry., Grand Rapids, Mich.	Jan. 2, 1900
WILSON, PERCY HARTSHORNE 211½ Market St., Camden, N. J.	Jan. 2, 1900

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	Date of Resignation.
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DODGE, RICHARD DESPARD	Dec. 31, 1899
HARISON, RICHARD MORLEY	Dec. 31, 1899
SCOTT, ARTHUR HILLMAN	Dec. 31, 1899

JUNIORS.

HORTON, THOMAS OSBORN	Dec. 31, 1899
ROBINSON, HENRY HOLLISTER	Dec. 31, 1899

DEATH.

MOFFET, JAMES DAVID..........Elected Associate Member Nov. 4th, 1891; Member Feb. 7th, 1894; died Nov. 3d, 1899.

Vol. XXVI. JANUARY, 1900. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXVI. JANUARY, 1900. No. 1.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE FOUNDATIONS OF THE NEW CROTON DAM.

BY CHARLES S. GOWEN, M. AM. SOC. C. E.

TO BE PRESENTED FEBRUARY 21ST, 1900.

In 1883 the Legislature of the State of New York passed an Act (Chapter 490, Laws of 1883) creating the Aqueduct Commissioners of the City of New York.

The purpose of this Act was the immediate increase of the water supply of the city which, under the conditions then prevailing, had for some time been inadequate and inefficient. To this end it was planned to begin the construction of a new aqueduct and a large dam on the Croton River, the latter near to and above the site of Quaker Bridge, at a point about 4 miles below the old Croton Dam which had been in use since 1839. This new dam, it was reckoned, would increase the available storage by about 32 000 000 000 galls., and, if construction were begun immediately, could be put to practical use, in connection with the New Aqueduct, not long after the completion of the latter, the work of which was planned to continue at the same time.

The Aqueduct Commissioners began the construction of the New Aqueduct in the fall of 1884, but found a strong opposition, on the part of a few influential citizens, to the project of the dam. This opposi-

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

tion resulted in an indefinite delay in action on the part of the Commissioners, so far as the large dam was concerned, but they ordered the construction of a smaller dam and reservoir near the head waters of the East Branch of the Croton, at the Village of Sodom, early in 1888. This action reversed the original plans for the enlargement of the water supply which were to build the large dam and basin at first and with as much speed as practicable, and later to complete the ~~conversation~~ of all the storage capacity of the Croton Valley by Conservation building the smaller dams and reservoirs, of which the dam at Sodom was one.

In July, 1888, a new Board of Aqueduct Commissioners came into power. They found a steadily increasing demand for more water, and came to the conclusion that it was best to continue the policy of building the smaller dams and reservoirs already inaugurated by their predecessors, as, owing to the time which had lapsed (about 4 years), without action relative to the proposed large dam, it was impossible, even by taking immediate action toward its construction, to complete it in time to afford the desired relief in the water supply. They, therefore, ordered the construction of the Carmel Dams (Reservoir D) and the Titicus Dam (Reservoir M), as well as the completion of the Sodom Dam System and Reservoirs, which included two dams, two reservoirs and a connecting tunnel. The construction of these works was started as soon as practicable, and further investigations were authorized in relation to the proposed large dam, in order to determine whether the best available site had been found.

To this end an extensive series of diamond-drill borings was made along the valley of the Croton River from the site of Old Croton Dam to a point nearly at the mouth of the river, about 1 mile below the old Quaker Bridge site. The result was the decision of the Commissioners, in January, 1891, to build the large dam at the Cornell site, a point about 1½ miles above Quaker Bridge, and so situated as to store nearly as much water as would have been stored by the Quaker Bridge Dam. The amount of storage by the dam if built at the Quaker Bridge site is estimated at 32 000 000 000 galls.; at the Cornell site, 30 000 000 000 galls.

In connection with these new dams and storage reservoirs are various older dams and natural lakes, throughout the water-shed of the Croton, which have been in use for the city's water supply for many

years, in connection with the Old Aqueduct; and the total storage capacity, upon the completion of the New Croton Dam, will be as follows:

Total storage in connection with the old works, including Central Park, Boyds Corners and Middle Branch Reservoirs..	9 541 000 000	galls.
Amawalk Dam	7 000 000 000*	"
Reservoir I, Sodom and Bog Brook Reservoirs	9 028 000 000	"
Reservoir D, Carmel	9 000 000 000*	"
Reservoir M, Titicus	7 167 000 000	"
New Croton Dam Reservoir	30 000 000 000*	"
Jerome Park Reservoir	1 500 000 000	"
	73 236 000 000	"

As the large reservoirs within the city territory cannot be emptied below certain limits without impairing the supply, the available storage capacity may be stated as about 70 000 000 000 galls.†

The construction of the New Croton Dam was begun in October, 1892, the contract for its construction having been let the preceding August. At the present time it is about two-thirds completed, and, as a general description of the structure, embodying its main features, is essential to the purposes of this paper, the following extracts from the "Report of the Chief Engineer, A. Fteley," Past-President, Am. Soc. C. E., "to the Aqueduct Commissioners, 1887 to 1895," are reprinted here, as they seem to embody the main points and important features in comparatively few words.

"The New Croton Dam at Cornell Site which is to form the largest reservoir of the system, on the lower part of the Croton River, was begun in October, 1892. It is located about 3¼ miles above the junction of the Croton with the Hudson, and about 1 mile above Old Quaker Bridge. The course of the Croton at this point is approximately west.

"At the dam location, rock (gneiss) crops out at the surface on the north side of the river, rising with a steep slope, which terminates at the top of a hill about 300 ft. high. The bed-rock on the north side dips quickly just before reaching the bank, and soundings show it at about 75 ft. below the river-bed. At this point, on a line about

* Approximate.

† Report of the Chief Engineer to the Aqueduct Commissioners, 1887–1895, p. 82.

parallel to and under the river, the rock changes abruptly from gneiss to limestone, with no marked change of surface level. The limestone extends across the valley at about the depth noted above, with some deeper pockets, and then rises gradually on the south side with the earth slope and below it, at varying depths, to a depth of 20 ft. at the extreme south end of the dam location.

"Under the river-bed the material above the bed-rock is largely sand, gravel and boulders. Approaching the south side of the valley, very compact hardpan and gravel next to the rock is indicated. The hardpan is surmounted next to the surface by a considerable layer of sand at the steep part of the slope. Along this slope, at about elevation 153 runs the Old Croton Aqueduct. The total distance across the valley at flow-line (elevation 200) is about 1 300 ft.

"The general features of the dam may be noted as follows:

"*An overflow, or spillway,* on the rocky side-hill forming the north slope of the valley.

"*A masonry dam* built on bed-rock and extending from its junction with the overflow at about the foot of the north slope of the valley, across and well into the south slope, where it ends in a wing-wall and core-wall for the embankment.

"*An embankment* with a core-wall extending to bed-rock from the end of the masonry dam up and along the south slope until elevation 220, the proposed top of this part of the dam, is reached.

"*The overflow* varies in height from 150 ft. at its junction with the main dam to about 10 ft., where it joins the side-hill at the upper end. This overflow runs along the side-hill and nearly parallel to the slope contours, curving up-stream at its junction with the masonry dam. The down-stream face of the overflow is to be formed in steps. From the spillway the water is to fall into a channel cut into the rock of the side-hill, through which the water will pass on its way to the river-bed below the dam. This overflow channel is to be about 50 ft. wide at the upper end and 125 ft. wide next to the main dam. The length of the overflow will be nearly 1 000 ft., elevation of top, 196.

"*The masonry dam* will extend from bed-rock to elevation 210, and provision is made for a roadway on top, 18 ft. in width. At the north end, near its junction with the overflow, is to be a gate-house of three chambers. Grooves in the masonry of the up-stream face will be provided for stop-planks, and in each chamber will be gates worked from the top of the dam, connecting with a 48-in. pipe. The pipes will extend through the dam, ending in a vault, containing stop-cocks to further control the flow of water. It is expected to place the lower openings in the gate-chambers at about elevation 75, nearly 30 ft. above the original river-bed, and to fill in this interval with earth, forming an embankment with a flat slope above the restored original surface, on the up-stream side.

"The masonry dam will be about 600 ft.* in length from its junction with the overflow to the back of the wing-wall at the south end, and its extreme height will be 260 ft. or more, as the soundings show some large and deep depressions in the rock surface below. Maximum thickness at bottom next to rock, about 190 ft.

"The embankment extending south from the wing-wall end of the masonry dam will have a core-wall extending throughout its length, founded on bed-rock, thus forming with the overflow and main dam a continuous masonry connection with bed-rock throughout the whole length. From elevation 64 down to bed-rock this wall is to be not less than 18 ft. in thickness; from elevations 64 to 200 the wall gradually diminishes to 6 ft. in width at the top. The elevation of the top of the embankment is 220; width at top, 30 ft. Up-stream slope, 2 to 1, paved; down-stream slope, 2 to 1, broken with three berms, each 5 ft. wide at different elevations. These berms will be ditched and paved to carry rain-water from the slopes, which are to be soiled and sodded.

"The Old Aqueduct is discontinued between the slope lines of the embankments, and is being replaced by a new section built on a curved line into the side-hill, nearer the extreme south end of the dam. At the junction of this new line of Aqueduct with the core-wall masonry, a second gate-house will be built for the purpose of connecting the water impounded in the New Reservoir with the Old Aqueduct.

* * * * * * *

"The gate-house foundation rests on bed-rock, and the curved line of the new section of the Aqueduct was designed to avoid the deep excavation for this foundation, which would have been necessary had the original location on the Old Aqueduct line been adhered to. The gate-house is drained by a system of 12-in. pipes, which are connected with the bottom of each chamber and unite into one pipe laid under the invert of that part of the new section of the Aqueduct lying on the down-stream side of the core-wall. Near the junction of the New Aqueduct Section with the Old Aqueduct, this drain pipe, after a short turn, emerges in the adjacent hillside.

"The center of the overflow and masonry dam, the core-wall, the gate-house foundations, the side walls of the Aqueduct, the backing of the gate-house chambers and inlet conduits will be built of rubble masonry. The overflow will be faced above the surface of the ground with coursed facing-stones cut to specified rises. On the down-stream side the steps are to be laid with block-stone masonry generally heavier in rise and width than the facing-stone, and of depth sufficient for a bond under the next step above.

* * * * * * *

* This length has since been increased to 710 ft.

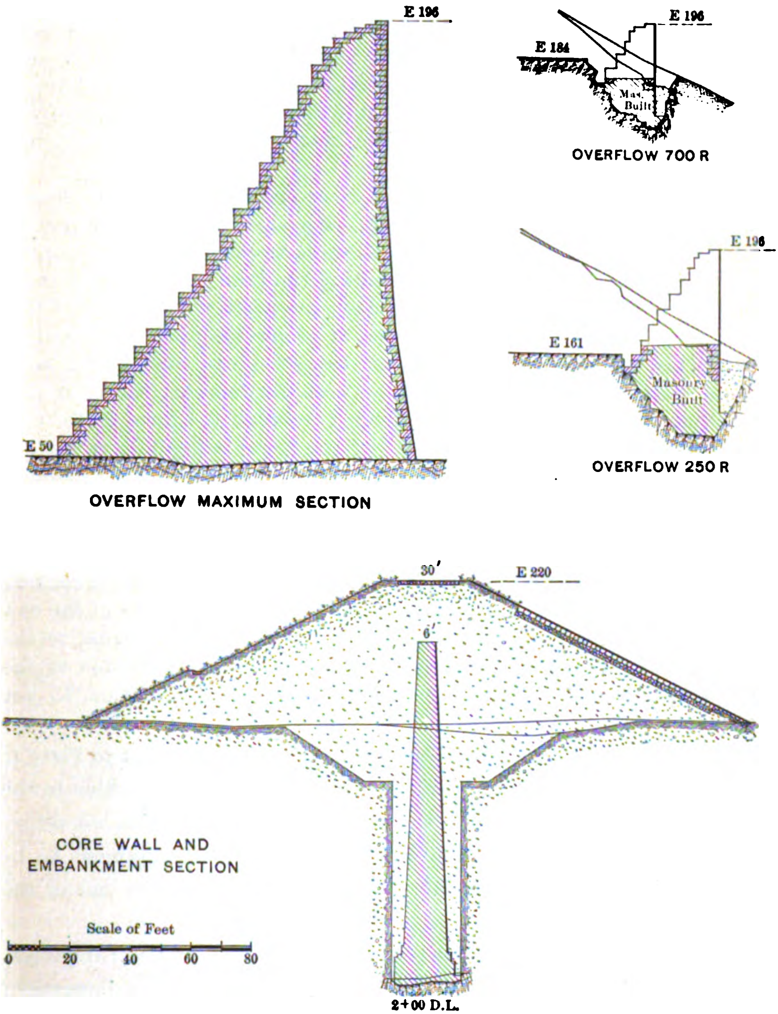

E 196
E 50
OVERFLOW MAXIMUM SECTION
E 196
E 184
Mas. Built
OVERFLOW 700 R
E 196
E 161
Masonry Built
OVERFLOW 250 R
30′
E 220
6′
CORE WALL AND EMBANKMENT SECTION
Scale of Feet
0 20 40 60 80
2+00 D.L.

Fig. 1.

"The main dam and the outer faces of its gate-house will be faced, wherever exposed, with facing-stone of the same class as that in the overflow.

* * * * * * *

"For the protection of the deep earth excavation, which is to take place in the bottom of the valley for the foundation of the dam, the river is diverted from its bed for a distance of over 1 100 ft. For that purpose an extensive rock cut has been made into the north side-hill and the river has been turned into this new channel" (125 ft. in width) "which is formed on the river side by a substantial river-wall founded in rock.

"This wall, parallel with the old river-bed and 600 ft. long, is connected at both ends with temporary wing-dams extending across the valley, above and below the site of the dam, thus making a complete inclosure, inside of which the excavation can take place without interference from the river. The wing-dams are built of earth with a timber core formed of two thicknesses of plank tongued and grooved, each 3 ins. in thickness. The timber core extends to a depth of 20 to 25 ft. below the natural ground. The toe of the dams on the river side is formed by heavy crib-work, intended to break the force of the current in time of freshet. The toe of the lower wing-dam is further protected by sheet piling and by a heavy weight of rock to counteract the erosive action of the large flow which may be discharged from the new channel into the river in case of a heavy freshet.

* * * * * * * *

"The total length of the protective work just described, including the river-wall and the wing-dams, is 1 600 ft. The capacity of the new channel has been designed to safely accommodate a flow equal to the largest freshet recorded in Croton River since the construction of the old works, when the discharge was approximately 15 000 cu. ft. per second."

In connection with this description, attention is called to Plate I, which is an outline plan of the structure and shows, in addition to the various features noted above, the outline of the excavation necessary for the main dam foundation masonry, and the embankment to be built against the core-wall with which it forms the south end of the structure.

Figs. 1 and 2 show various sections of core-wall and embankment, of the main masonry dam at various points and the maximum section of the overflow wall where it crosses the temporary river channel.

The dam was designed and its construction is being superintended by Mr. Fteley, the Chief Engineer. He was assisted, for the mathematical computations necessary for determining the main section, by

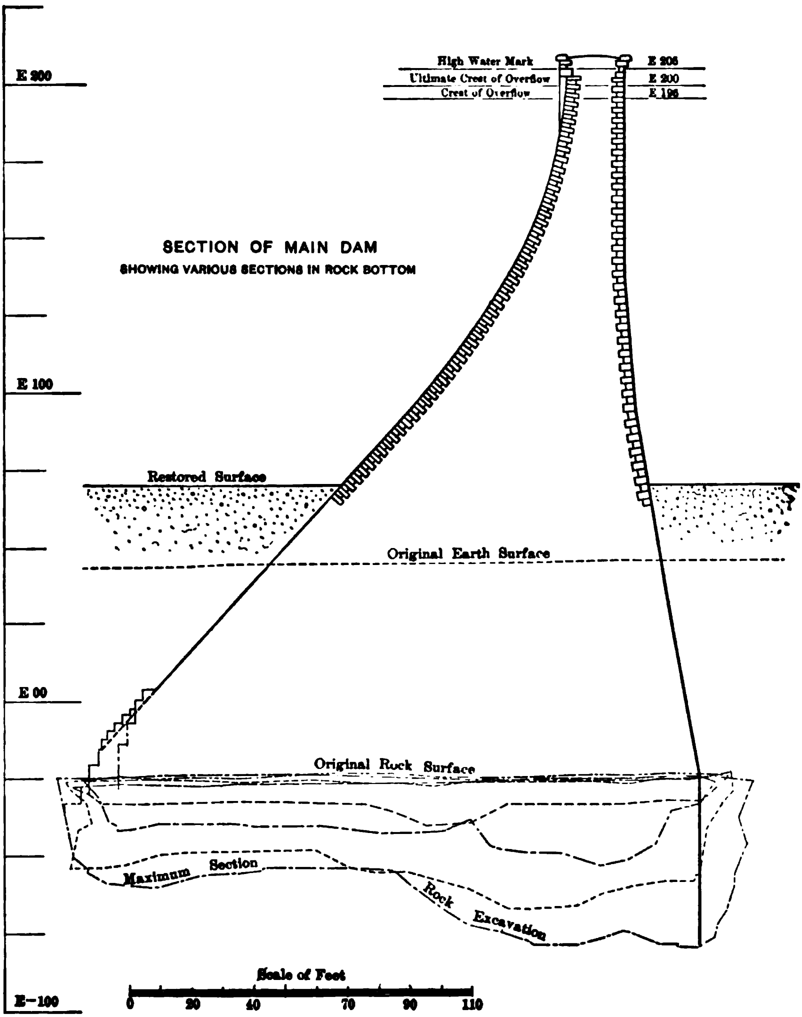

High Water Mark
Ultimate Crest of Overflow
Crest of Overflow
E 200
E 200
E 100
E 00
E-100
SECTION OF MAIN DAM
SHOWING VARIOUS SECTIONS IN ROCK BOTTOM
Restored Surface
Original Earth Surface
Original Rock Surface
Maximum Section
Rock Excavation
Scale of Feet
0
20
40
70
90
110

Fig. 2.

E. Wegmann, M. Am. Soc. C. E., who has since developed and formulated the methods followed, in his book on high masonry dams.*

It may be said that the section adopted affords a factor of safety of 2 against any tendency toward the overturning of the structure.

The work of construction has been conducted, from its inception, under the immediate direction of the writer.

Since the foregoing description was written, the protective work has been completed, substantially as outlined. The earth and rock excavations for the foundations have been finished; the foundation masonry practically all laid, excepting a short stretch of the overflow which is to cross the river-channel cut and join the long stretch of overfall foundation masonry already laid. The length of this remaining stretch is about 250 ft. In the progress of the above work the section of the main dam masonry was carried about 110 ft. further into the side-hill at the south end than was planned at first, thus shortening the core-wall and embankment section by the same distance, and, owing to the rise in the bed-rock surface under the south slope, decreasing the maximum depth or height of the core-wall and embankment considerably from that of the original design.

Owing to the character of the limestone, which rendered deep excavation necessary at certain points, the extreme height of the masonry dam will range from Elevation—80, the lowest point reached in the foundation excavation, to Elevation 210, a total of 290 ft. For the same reason the extreme thickness of the main dam masonry at the toe is about 200 ft.

Borings.

The final location of the New Croton Dam resulted from the indications furnished by an extensive series of diamond-drill borings, during which the Croton Valley was explored thoroughly along the line of the river from an old mill at the head of tide water, about three-quarters of a mile below Quaker Bridge, to Old Croton Dam, a distance of about 5 miles. The general system for determining upon the position of the borings proposed, was as follows: Whenever the appearance of the surface seemed to be favorable a number of drill holes were made on a line parallel with the river, and, if one of them gave indication of the proximity of bed-rock to the surface, a transverse line of holes was drilled across the valley at that point.

* "High Masonry Dams," by E. Wegmann, Jr., M. Am. Soc. C. E., New York, John Wiley & Sons, 1891.

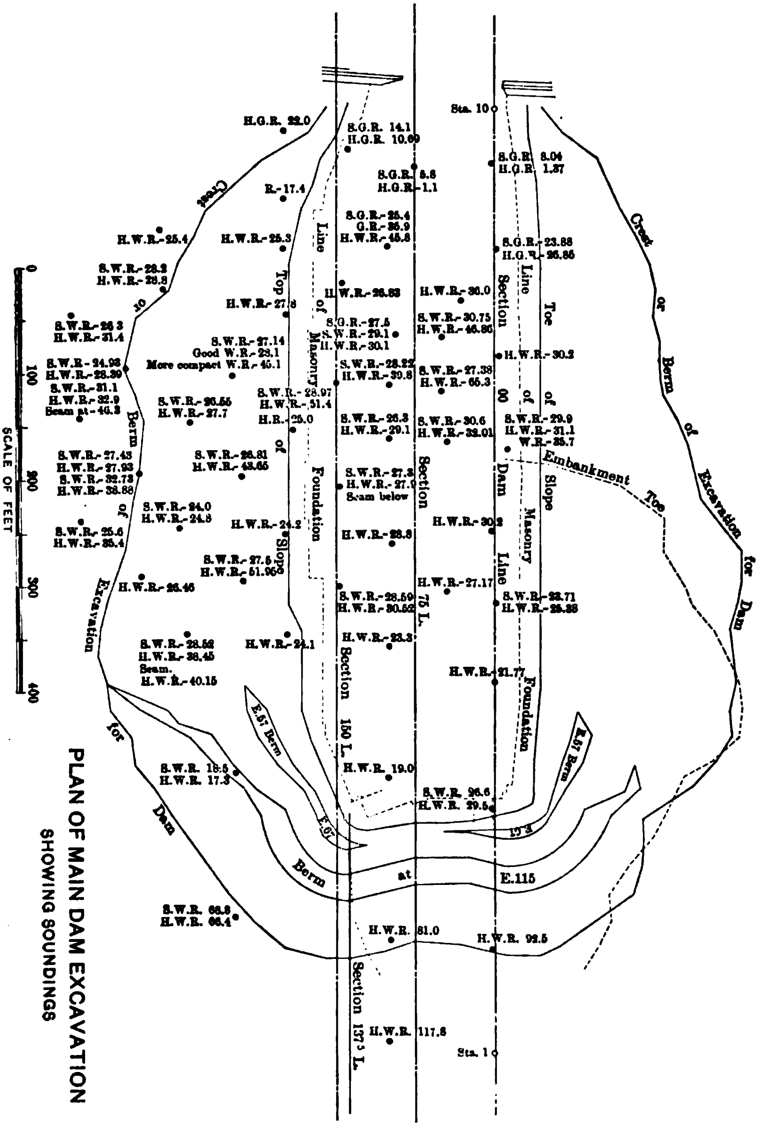


FIG. 8.

PLAN OF MAIN DAM EXCAVATION
SHOWING SOUNDINGS

In this way a large number of transverse lines was drilled, and it was found almost invariably that wherever the bed-rock cropped to the surface on one side of the valley it dipped down sharply on the other side to a depth at which, in most cases, it would be impracticable to establish a foundation. As a rule, gneiss was found, but at various points on either side there are formations of limestone with clearly defined points of separation which, in some cases, were under the bed of the stream.

Several of the more favorable locations thus indicated were explored more particularly by a number of transverse lines of holes about 100 ft. apart, and when the present established location was finally determined upon, additional borings were made, to cover the site of the masonry structure, at intervals of 50 ft. Plate 3 shows the location and result of these borings, as well as the outline of the proposed foundations. It will be seen that there were in places indications of a considerable depth of soft white rock (partly disintegrated limestone), before the hard rock was reached, extending in one case to an extreme of nearly 40 ft. The holes drilled in the rock were, as a rule, 2 ins. in diameter, and were carried to a depth presumably sufficient to establish the character of the rock below. The hard white rock sought for, and, as a rule found before the borings ceased, was mostly bluish limestone, while the soft white rock varied in its texture from white limestone, friable under some pressure, to very friable or wholly decomposed rock. The line of separation between the limestone and the gneiss was shown to be directly under and parallel to the river-bed. The borings indicated further, the presence of seams, more or less open, in the limestone, and the frequent reports of the sudden loss of the water (*i. e.*, the water supplied by the steam pump to wash out the holes as the borings progressed) showed that these seams were connected in places with rather free flowing outlets. As the general level of the bed-rock was at Elevation —25, or about 75 ft. below the river, and as the water table in the sand and gravel above this bed-rock was substantially the same as the river level, it is perhaps a question of some interest as to how and where this disappearing stream went, and, in case of its reappearance, what were the causes which may have led to it. Copies of the drill runner's log, which follow, show the records of Holes Nos. 95 and 99.

These furnish two illustrations out of a number of cases in which the water disappeared and reappeared after an interval. Hole No. 99 is especially noticeable, as the final disappearance of the water did not occûr until the drill had reached its lowest level, Elevation —76.80.

HOLE No. 95.—Elevation of Jack Plank........... 71.9
" " Ground............... 69.6

Date. 1892.		Material.*	Depth, below Jack Plank.	Remarks.
April	20.	S. & B.	7.23	
"	21.	"	27.50	
"	22.	....	....	Broke casing; third joint up 28.85.
"	22.	....	....	Pulled out; got back to 28.85.
"	22.	Boulder.	28.35	X bit 10 ins. below shoe.
"	22.	C. S. & B.	30.00	
"	25.	"	36.00	Don't stand up; fills in.
"	26.	"	43.25	" " "
"	27.	"	51.00	" " "
"	28.	"	56.00	" " "
"	28.	S. G. S.	58.00	Very little flow as soon as X bit is below shoe.
"	29.	"	59.00	Very little flow as soon as X bit is below shoe.
"	29.	H. S. & S.	74.00	Stands up and fills in; can pound down; stands up; no flow.
"	30.	"	79.00	Stands up and fills in; can pound down; stands up; no flow.
May	2.	"	91.00	Stands up and fills in; can pound down; stands up; no flow.
"	3.	"	94.00	Fills in very bad; cannot get powder down.
"	4.	"	98.67	Stands up good.
"	5.	"	100.87	—28.97 top of soft white rock.
"	5.	S. W. R. & Sand.	104.85	—32.95 I think this is fine sand; the floor was clear.
"	5.	S. W. R.	105.90	No core.
"	6.	"	106.90	Lost flow.
"	6.	"	110.00	Flow came back; no core.
"	6.	"	111.00	The rock is a little harder; no core.
"	6.	"	114.85	Not hard enough to core.
"	6.	"	118.85	" " " yet; no core.
"	6.	"	122.00	Lost flow.
"	6	"	122.95	Not hard enough to core; no core.
"	6.	"	123.30	Commenced to core —51.40.
"	6.	H. W. R.	124.55	0.90.
"	6.	"	126.95	0.90.
"	6.	"	130.25	1.80 Elevation of water in casing x 45.9.

* S. & B. —Sand and boulders.
C. S. & B.—Coarse sand and boulders.
S. G. S. —Sand, gravel, stones.
H. S. & S.—Hard sand and stones.
S. W. R. —Soft white rock.
H. W. R. —Hard white rock.

"This hole is the same as Hole No. 88; stands up very good, but could not go far below the shoe, the flow would go away. The rock from elevation —28.97 —51.40 was very soft, but stood up very good, and did not cave in, if it had I could not have drilled so far down. W. J. S. (Signed) Tierney, Foreman."

HOLE No. 99.—Elevation of Jack Plank............72.2
" " Ground...............70.0

Date. 1892.		Material.*	Depth, below Jack Plank.	Remarks.
May	19.	S. & S.	15.00	Moved, set up, down to 15.00.
"	20.	F. S. & S.	36.50	Loose fine sand and no flow.
"	20.	C. S. & S.	42.86	Stands up good, flow came back.
"	20.	"	51.00	" " "
"	23.	"	53.00	" " "
"	23.	S. G. & S.	64.50	Telescoped with 4-in. casing to 38.50.
"	24.	"	73.00	" 2½-in. "
"	25.	"	79.00	Fills in bad.
"	26.	"	83.00	"
"	27.	"	91.50	"
"	28.	"	95.00	Stands up good, very stony.
"	30.	"	97.50	" "
June	1.	"	99.58	Top of S. W. R. —27.38.
"	1.	S. W. R.	100.80	
"	1.	"	103.0	Put in diamond bit.
"	1.	"	104.9	No core.
"	1.	"	108.4	"
"	2.	"	114.1	"
"	2.	"	121.8	"
"	2.	"	125.9	"
"	2.	"	128.2	"
"	2.	"	130.5	Commenced to core —58.30.
"	2.	H. W. R.	131.6	0.60 core.
"	3.	S. W. R.	133.6	No core.
"	3.	"	137.50	No core. (—65.3).
"	3.	H. W. R.	140.40	1.70 core, commenced to core —68.2.
"	3.	"	142.00	0.60 "
"	3.	"	143.50	0.40 "
"	6.	"	145.50	0.25 "
"	6.	"	147.50	0.60 " Lost part flow 147.8.
"	6.	"	148.90	0.75 "
"	6.	"	150.15	0.60 " Lost all flow 149.0.
				" M. TIERNEY."

*S. & S. —Sand and stones.
F. S. & S.—Fine sand and stones.
C. S. & S.—Coarse sand and stones.
S. G. & S.—Coarse gravel and stones.
S. W. R. —Soft white rock.
H. W. R. —Hard white rock.

" June 2d. Put in diamond bit at 103.0. Commenced to core at 130.50; rock was not soft like mush; could not turn rods down with the tongs, but was not hard enough to core; did not find any seams or soft spots; stood up good; did not fill in. June 3d, no seams, no soft spots, but not hard enough to core. X Rock in Hole 99 was hard enough to stand up but did not core. Did not find any soft mushy seams. Commenced to core —58.3, cored to —59.40, hard did not core until I got to —68.20. Then I picked up some core all the way down, as report will show, lost part flow —75.60. Lost all flow —76.80. W. J. Sager."

Fig. 4, 5 and 6 are three sections of the foundation rock on which the main dam is built. The limits of hard and soft rock surface, as indicated by the soundings, as well as the actual surface exposed upon excavation and the actual surface built upon, are shown. These sections are interesting as a comparison between the possible results, as

shown by the diamond-drill work, and the actual results obtained. In a general way, it may be said that the rock was found to be more broken up and traversed by seams, fissures and soft streaks, in all the various conditions exhibited by limestone ledges, than might have been expected from general surface indications in the neighborhood and from the borings themselves. To a certain extent, the same was true of the gneiss, the surface of which was found to be full of slips and seams running in every direction between hard masses, while extensive pockets and seams of disintegrated rock of considerable

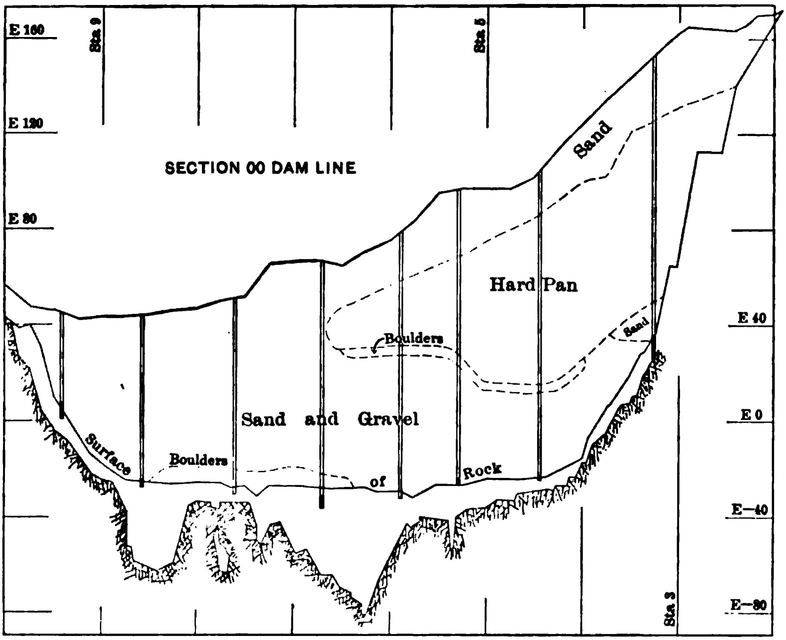


FIG. 4.

width had to be removed or excavated until, in the case of the seams, which were mostly nearly vertical, they narrowed up and nearly or quite pinched out.

The following statement, quoted from the "Report of the Chief Engineer" Mr. Fteley, to the Aqueduct Commissioners, 1887 to 1895, is given here in explanation of the fact that it was finally decided to build the dam at this point, although at the time the decision was made all facts in connection with this location had not been devel-

oped, and its superiority to other sites was still an open question, while the additional borings, made, as previously noted, after the site had been decided upon, showed no more encouraging results at least than those made earlier.

"No very favorable location was found, and the writer reported to the Aqueduct Commission on October 8th, 1890, that it would be advisable to abandon for the present the Quaker Bridge site, and to build a dam of less magnitude a short distance below the present Croton Dam (see Location 2, Line C, on Sheets 27 and 29). Although the reservoir to be thus formed would have contained an available

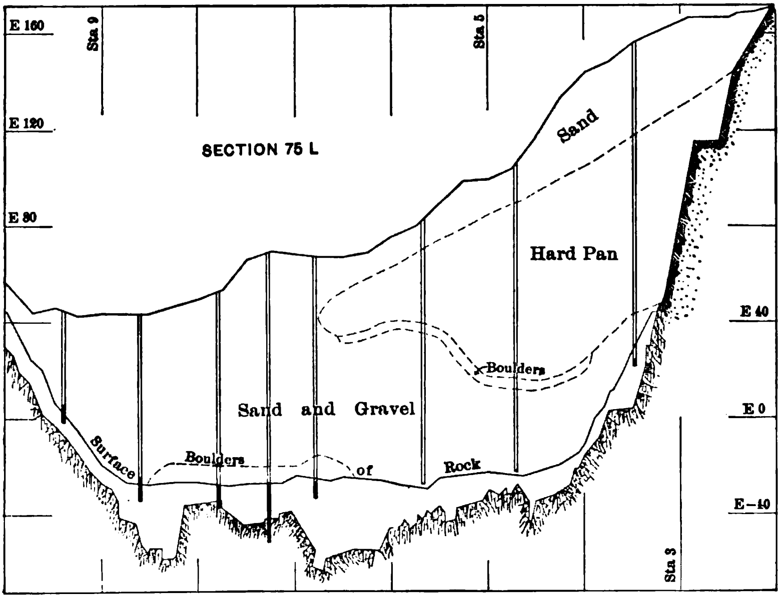


FIG. 5.

storage of about one-half that of the Quaker Bridge Reservoir, the principal reasons given in favor of that opinion were:

"*First.*—That the storage to be thus obtained would be sufficient for many years to come.

"*Second.*—That the height and cost of that dam would be much less, and that it could be built in less time.

"*Third.*—That the experience which would soon be acquired of the effect of the large storage reservoirs under construction on the quality of the water, would better enable the authorities in charge to ascertain whether it would be of good policy in the future to build the higher dam or to resort to some other mode of increasing the supply.

"*Fourth.*—That the interest of the money thus saved for the present would, after twenty-five years, represent a large part of the money necessary to then build the higher dam, with the result that the city would then have two dams instead of one for nearly the same expenditure.

"The report also mentioned that another site (the Cornell's site), not then fully explored, presented good features and should be further considered.

"The Aqueduct Commissioners voted to adopt the last-mentioned site, which is one mile and a quarter above Quaker Bridge.

"Borings made subsequently to this decision disclosed that the rock strata, at places, were found to be at a greater depth than was anticipated; hence, the excavation will be deeper than was originally intended, and the bulk of masonry will be correspondingly larger."

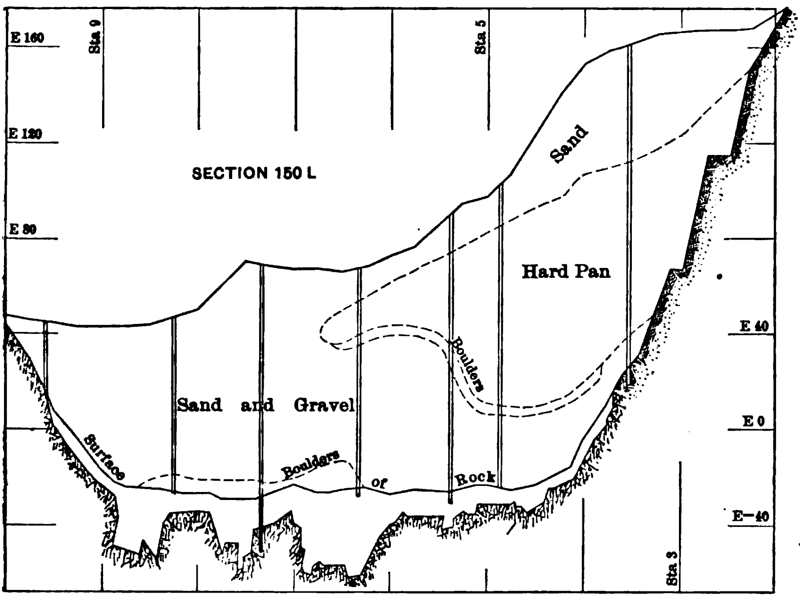


FIG. 6.

PROTECTIVE WORK.

Plate I shows the general plan of the protective work designed and built for the purpose of enabling the deep excavation necessary for the main dam foundations to be carried on with the smallest chance of interruption from floods.

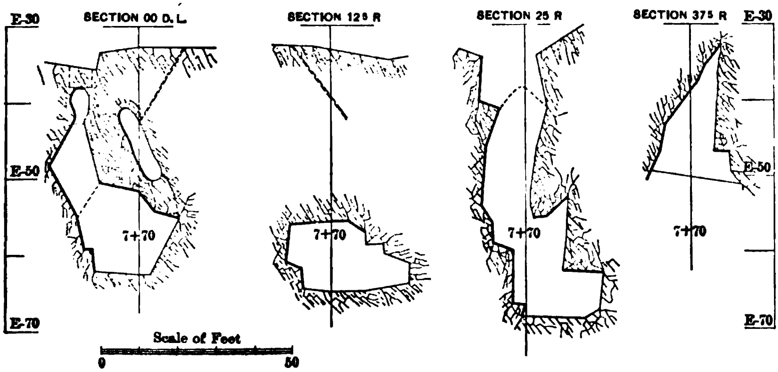
The section of the river wall which separates the new river channel from the main excavation is shown by Fig. 7. This wall is 600 ft.

long, and is built on the underlying gneiss, care being taken, throughout its length, to make its bond with the foundation rock as complete as possible, particularly throughout that portion which comes within and forms a part of the main dam. Throughout this section, excavation was made in the foundation rock to a considerable depth to get below open seams and fissures, and during its construction a considerable portion of the foundation of the main dam overlying the channel cut was laid up to the grade of the channel, advantage being taken of the necessary foundation work of the adjacent river wall to do it.

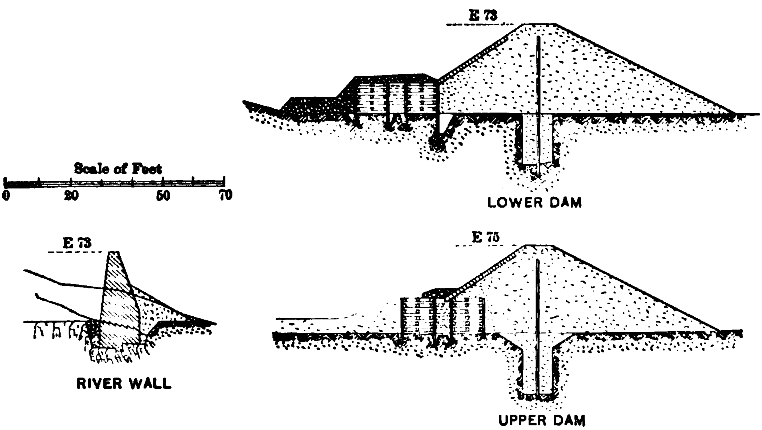
Fig. 7 shows also sections of the upper and lower earth wing-dams as built, and their position, relative to the main excavation cut, is shown in Plate I. The main lines of 3-in. sheeting, which were relied upon as the water stops, were carried down from 20 to 25 ft. below the original surface. As most of the material in which this sheeting was placed was coarse, loose gravel and sand, resort was had to trenching, with sides temporarily sheeted, and the permanent sheeting, after being placed in position, was driven by heavy hand mallets down an additional foot or two. At the east end of the upper wing-dam, however, for a considerable distance, the bottom was found to be of quicksand, and the sheeting was put down, through a considerable part of the depth reached, by means of a water-jet and heavy hand mallet.

The crib-work is designed to protect the embankment toes from the great erosion to be expected in case of a heavy freshet, while the extra sheeting and loading of stone on the lower wing-dam crib is a still further protection against the wash of the discharging channel, which, in extreme cases, might be strong enough to displace the loading and possibly cause a slight movement of the cribs which, in such cases, are so planned as to yield measurably outward without materially endangering the toe of the embankment.

While the river channel and these dams are designed to carry in emergency 22 ft. of water, or more, it may be said that at the present writing the deepest flow experienced through the channel has been about 11 ft. This was due to a warm rain of 3.6 ins., most of which fell in about 12 hours, on 3 ins. of snow lying on deeply frozen ground, in the month of February. From this it is easy to see that a combination of circumstances resulting in a flow which would tax the channel to its full capacity is quite possible.



SECTIONS OF CAVE



SECTIONS OF WING DAMS

FIG. 7.

Before leaving the subject of the protective work, attention is called to the somewhat extensive and perhaps seemingly permanent character of its design and construction. This work involved, in the construction of the river channel, an earth excavation of about 100 000 cu. yds., and rock excavation of about 106 000 cu. yds. The river wall and wing-dams include in their construction:

Earth Excavation	8 500 cu. yds.
Vertical Trench Excavation	6 700 " "
Refilling and Embankment	58 000 " "
Rock Excavation	4 000 " "
Timber	390 000 ft. B. M.
Crib Work	7 000 cu. yds.
Rubble Masonry	10 000 " "
Paving and Rip Rap	2 000 " "

While the cost of the above work is a large amount (upwards of $350 000), its proportion to the total cost of the dam, which may amount to $5 000 000, is not excessive, and it must be remarked that a considerable portion of it will form a part of the permanent structure. It seems to have been justified on account of the very efficient protection it has afforded to the extensive excavation work, both of earth and rock, and the foundation masonry work, which have been carried along steadily for three years and which, in the case of the masonry and refilling, must continue for another year at least before the dams will cease to be necessary. The extreme depth of the pit, in which the work has been done, below the river bed, is 130 ft.

Earth Excavation, Main Dam Foundation.

This work involved preparation for a foundation on rock extending from about Station 3 + 30 to about Station 10 + 00, where the new river channel, formed in connection with the protective work, is merged into the foundation, and which varies in width from about 200 ft., at the lowest point, to about 130 ft. at Station 10 + 00 and 140 ft. at Station 3 + 30, on the line of the back of the proposed wing-wall (see Fig. 3). The necessary earth excavation covering this area was about 885 000 cu. yds., consisting largely of loose sand, gravel and boulders with, however, at the south end of the pit, a large area of hardpan excavation, this hardpan forming, to a considerable extent,

PLATE I.
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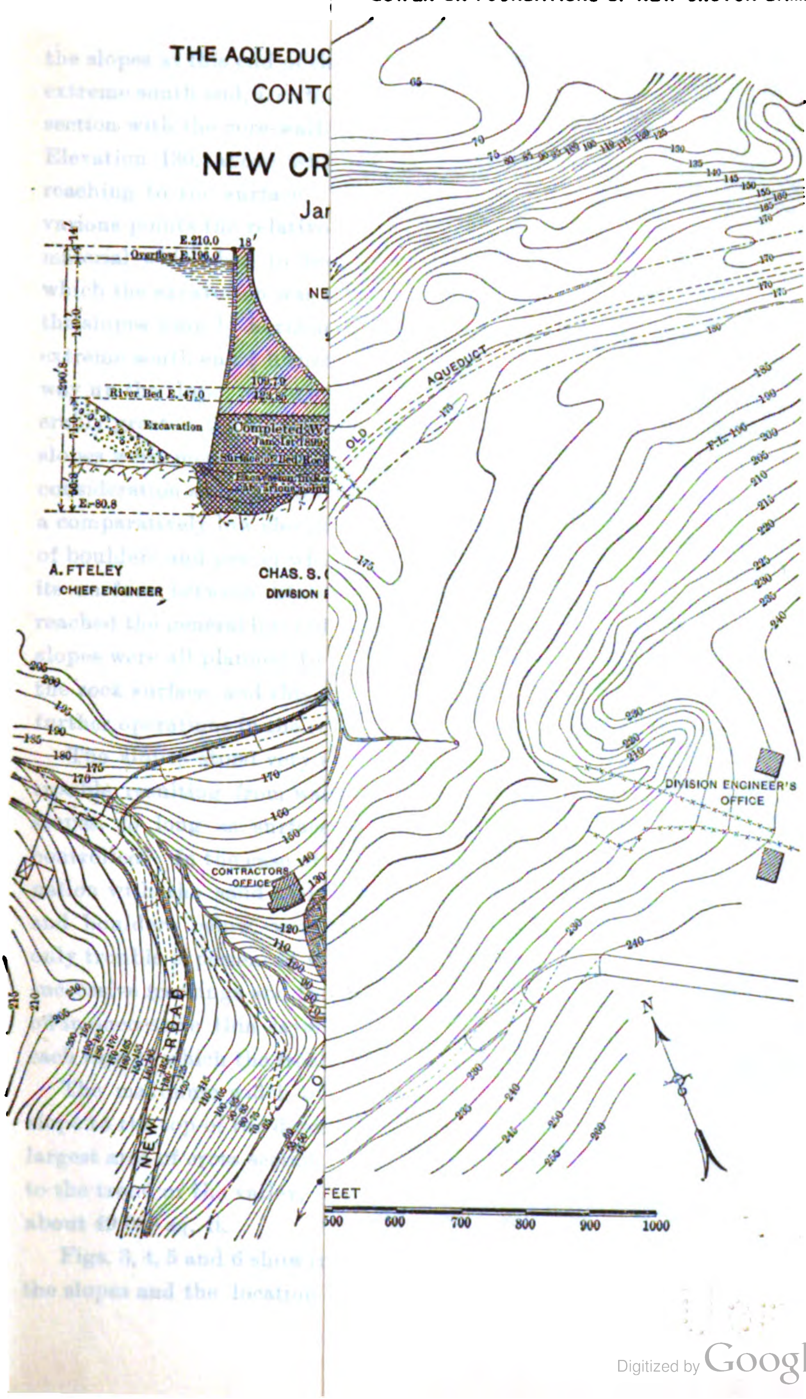
JANUARY, 1900.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



the slopes at this end of the excavation, and extending in depth at the extreme south end, *i. e.*, the point of junction of the main dam masonry section with the core-wall, from the surface of the bed rock to about Elevation 130, above which it was surmounted by loose, fine sand reaching to the surface. Figs. 4, 5 and 6 are sections indicating at various points the relative positions of the different kinds of earthy material which had to be moved, and the south end slope lines to which the excavation was made. In the case of the gravel and sand, the slopes were 1½ horizontal to 1 vertical, and in the hardpan at the extreme south end ½ horizontal to 1 vertical, with a berm about half-way up the slope; while on the quarters, where the depth was considerably greater, the slopes and berms were varied somewhat, as the end slopes were merged into the side slopes. In laying out the slopes, consideration also had to be given to the fact that on the quarters, at a comparatively low elevation, the hardpan was underlaid with layers of boulders and gravel which extended to the bed rock as it dipped in its surface between Station 3 + 30 and Station 5 + 00, where it reached the general level of the rock in the valley bottom. These earth slopes were all planned to allow for a toe berm of 20 ft. in width, at the rock surface, and this space proved to be necessary and useful in further operations in the rock bottom below.

The slopes stood very satisfactorily, on the whole, no particular trouble resulting from washing or sloughing, in case of the gravel slopes, so long as surface drainage outside the pit was properly controlled. In the case of the hardpan, steep slopes which in combination with the sand above and at certain points with sand, gravel and boulders below, were, at the maximum, 150 ft. in height, the only trouble experienced was during the open winter of 1897–98, when successive freezings and thawings caused the slope surface to slough off in successive thin layers representing in thickness the depth in each case to which the frost had penetrated since the preceding thaw.

The maximum width of the pit, from the top of the up-stream slope to the top of the down-stream slope, was about 600 ft., and the largest area of cross-section excavation, above bed rock and parallel to the trend of the valley, *i. e.*, at right angles to the dam line, was about 49 000 sq. ft.

Figs. 3, 4, 5 and 6 show in plan and section the crest and toe lines of the slopes and the location and elevation of the berms in the steep

slope at the south end of the pit. The line of the masonry foundation is also indicated and its connection with the core-wall. A section at 137.5 L., in Fig. 8, shows the ordered and actually excavated slope in the hardpan at its highest point.

A very large amount of this excavation, lying on the south slope of the valley and above the level of the river, was removed by steam-shovels, three of which were in use at one time. The first work done in sinking below the river bed was by means of a large "orange peel" dredge, specially constructed for the purpose and used for the excavation of the loose gravel and sand until the near approach in depth to bed-rock, and the necessity of beginning rock excavation, demanded a change in methods, as the dredge work was dependent upon a certain depth of water in which to work the bucket efficiently, while the rock excavation rendered close drainage necessary. For the further prosecution of this work resort was had for some time exclusively to three cable-ways stretched across the valley longitudinally along the line of the dam at such transverse intervals as to cover the plan of foundation. These cables were installed for the purpose of aiding the earth and rock excavation and, ultimately, for taking in stone and other material for the dam masonry. They were used for some time in connection with the dredge above mentioned, and were in turn supplemented, when the rock excavation work assumed large proportions and there was considerable earth work remaining, by railway inclines placed successively at different points on the side slopes and worked by means of stationary hoisting engines and cables.

With the use of railway inclines, steam-shovels were again operated, and a large amount of coarse indurated gravel, lying just above the bed rock at the north end of the main cut, was thus excavated, and, as the excavation progressed toward the south end of the cut and the hardpan was reached, it was removed almost wholly with the aid of heavy steam-shovels, although the slope trimming at the south end and on the quarters, and some bottom cleaning up on the rock surface, had to be done by hand with the aid of skips and derricks.

Fig. 1, Plate II, shows a part of the river wall and lower wing-dam, and the progress of the main dam excavation to September, 1895. The large pit shown was excavated mainly by means of the dredge, shown on the extreme right, with considerable assistance from the cableways, for which the material was excavated by hand into large

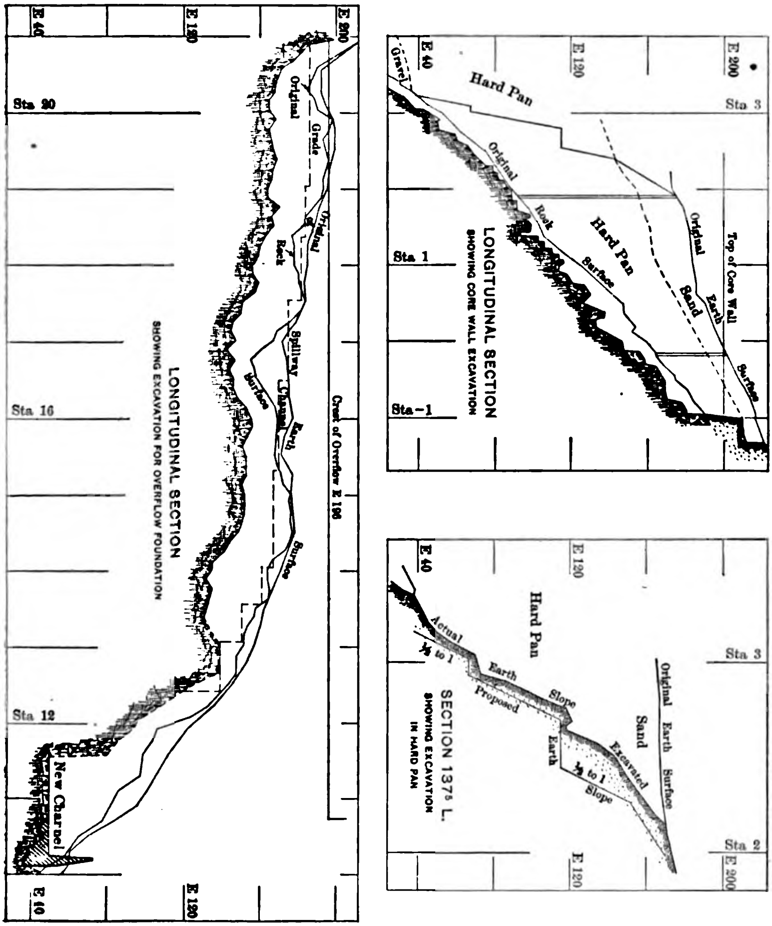

LONGITUDINAL SECTION
SHOWING EXCAVATION FOR OVERFLOW FOUNDATION
Crest of Overflow E 196
Sta 20
Sta 16
Sta 12
E 40
E 120
E 200
Original
Grade
Rock
Spillway
Channel
Earth
Surface
New Channel
LONGITUDINAL SECTION
SHOWING CORE WALL EXCAVATION
Hard Pan
Gravel
Sand
Top of Core Wall
Sta 3
Sta 1
Sta -1
SECTION 137⁵ L.
SHOWING EXCAVATION
IN HARD PAN
Actual
Proposed
Excavated
Slope
Sta 2

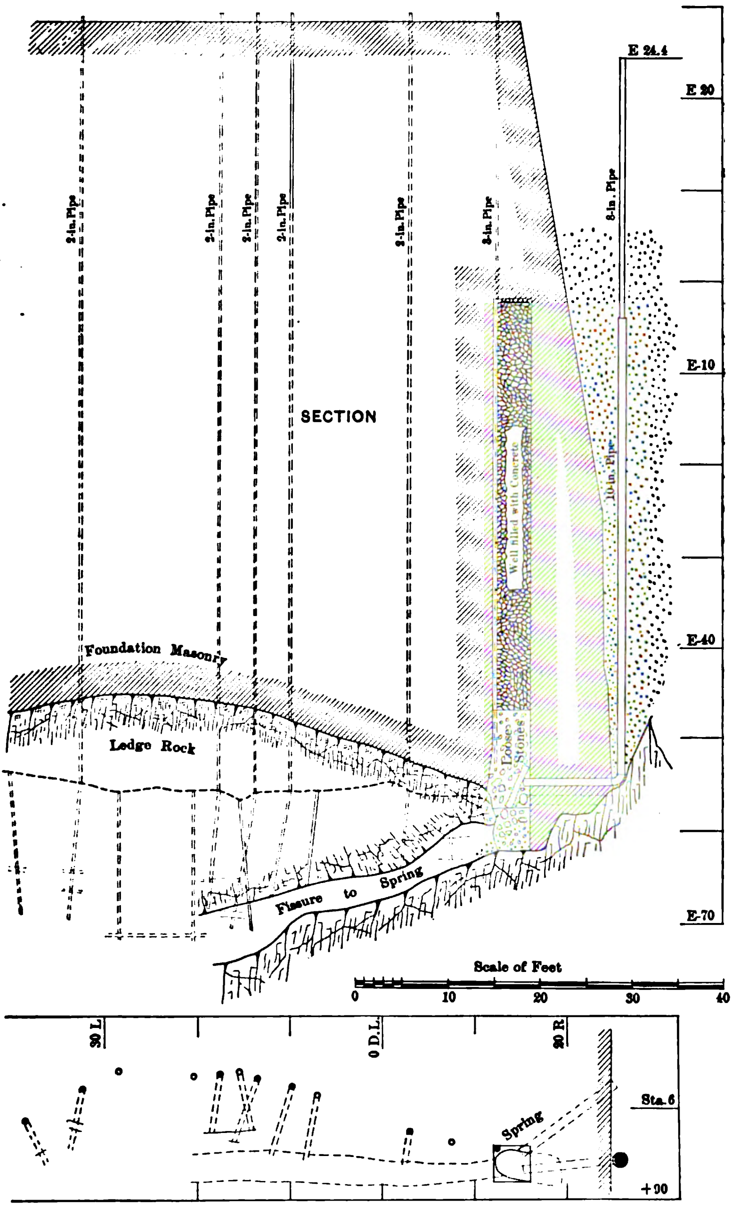
FIG. 8.

scale pans, and hoisted and transferred to the foot of the heavy slope shown in the rear, where it was dumped into cars. The levels above the pit, shown in Fig. 1, Plate II, were excavated with steam-shovels, and later, as further progress was made into the hard material of the great slope at the south end of the cut, steam-shovels were again placed at a lower level in the pit and the inclines were used as mentioned previously. Fig. 2, Plate II shows more particularly the steep slopes in the hardpan as finally shaped, and Fig. 9 shows in a more general way the side slopes, but at some time after the excavation was completed, when a small amount of back-filling had been done. Also, at this time, cuts had been made in the side slopes, forming berms on which side tracks were laid, to furnish supplies for the foundation masonry. Fig. 2, Plate II, shows particularly the hardpan slopes (½ horizontal to 1 vertical) and berms at the south end and on the quarters, as well as the underlying rock bottom excavated for the foundation masonry.

Core-Wall Excavation.

The core-wall extends from the south end of the main dam for a distance of about 430 ft. into the side hill. Its general section and the cross-section of the trench excavated for it are shown in Fig. 1. The maximum width and height of this wall, which occurs at its junction with the main dam masonry, are, respectively, 18 ft. and 175 ft. The material excavated for the wall was hardpan above the limestone foundation up to within a depth from the original surface varying from 24 ft. to 8 ft. Above this hardpan were gravel and sand. The general extent, as well as depth of excavation for this wall, together with the line limiting the top of the trench excavation, are shown on the profile, Fig. 8.

The trench walls were vertical, the sustaining power of the hardpan allowing the sheeting and bracing to be done after the completion of the successful levels excavated, which levels varied from 6 to 12 ft. in height or depth, according to the depth of the section of trench then under excavation. As stated, the hardpan, throughout the length of this trench, extended to the rock foundation, which showed considerable variation in hardness and texture, and called for excavation of considerable depths below the rock surface in certain places before compact layers of sufficient hardness were found. Fig. 1, Plate



PLAN
FISSURE AND SPRING AT STA 5+95
Piped holes shown thus •

FIG. 9.

III, shows the rock bottom ready for the masonry of the core-wall, as well as the sheeting and sides of the trench for a certain distance up, at Station 1 + 80, 150 ft. from its junction with the main dam. At this point the rock was sufficiently compact and of necessary bearing strength, although not very hard, and the steps shown in the inclined surface of this foundation were made with picks and shovels. The depth to which the rock was excavated varied from 4 to 7 ft.

The width of the trench is measurably greater than the thickness of the core-wall, and the difference was liberally planned in order that there should be no difficulty in finding working room at the bottom of the trench to remove the bracing and sheeting after the masonry foundations of the wall were started. It also gave proper opportunity to place the refilling; which was of the same material as had been excavated, and was placed very carefully in layers varying from 2 to 4 ins. in thickness, and thoroughly rammed by hand. Advantage was also taken of this extra width to widen the footing or lower courses of the core-wall, thus increasing the bearing surface in certain places where the rock foundation might possibly call for it, and the section shown in Fig. 1 is taken at one of these points.

As to the thickness of this wall, which it will be noted is somewhat massive, varying from 6 ft. in thickness at the top to 18 ft. at the lowest point, it may be said that the wall was purposely designed not only as a water-tight screen reaching from bottom to surface between the upper and lower sections of the enclosing embankment, but also to afford a substantial resistance to any overturning or crack-producing force which might be caused in the course of time by the saturation of the up-stream bank and its consequent increase of unit weight.

The maximum depth of sheeted vertical trench excavation, including the depth of excavation in the foundation rock, was 75 ft. This point was at Station 2 + 50. At this point the top of the vertical trench was 27 ft. below the original surface of the ground. The earth material above the core-wall trench level was excavated by steam-shovel; below, in the trench proper, it was excavated by pick and shovel, and removed by derricks. Black powder was generally used in sinking the trench, at the lower levels particularly, to loosen the hardpan, and it was used very extensively for the same purpose in the main cut, both for facilitating the work of the steam-shovels and for all handwork done in the removal of hardpan.

FIG. 1.—FEBRUARY 7TH, 1896. VIEW FROM UPPER END OF FOOT-BRIDGE, LOOKING S. E. FLOOD IN FOREGROUND.

FIG. 2.—OCTOBER 1ST, 1897. SLOPE AND BERM AT SOUTH END OF CUT.

Overflow Excavation.

At the present writing, the completed overflow foundations, embracing a length of 750 ft., extend along the side hill on the north side of the river and finally abut in the rock of the hill at the upper end.

This rock foundation is entirely country rock, or gneiss, and the amount of superimposed earth was not large, and was mostly sandy loam on the surface, with underlying gravel.

Fig. 8 shows a profile of the earth and rock excavation as well, and on Fig. 1 are shown representative cross-sections, indicating more clearly the extent to which rock excavation was found necessary to insure a fairly tight bottom. The rock was full of seams and faults, and considerable depths had to be reached at certain points in order that open seams running across the line of the structure might be followed until they pinched out. The extensive rock excavation in the front of this foundation work, shown in the cross-sections, was necessary to provide the waterway leading from the spillway bottom to the old river-bed below the main dam.

Rock Excavation and Foundation for the Main Dam.

As stated in the general description of the dam, the rock on the north side of the valley, on the steep side hill, cropped out at points very near the surface. It was formed of gneiss, considerably fissured, but generally sound after reaching a certain depth in the ledge. This gneiss extended to the line of the old bed of the river, where its depth below the surface was much greater, being about 75 ft. The section under consideration was found to be well broken up near the surface by open seams of considerable width, varying from 2 to 3 ins. in cases. Such seams were filled with earth, and extended in all directions. There were also some strata of rock, more or less disintegrated. These varied from 1 to 3 ft. in width or thickness, and were removable with pick and shovel for some depth from the surface. The dip and strike of this rock were about the same as that of the limestone beyond; the dip being nearly vertical and the strike following the line of the valley at right angles to the dam.

Under and beyond the river-bed, the character of the rock changes entirely, being composed wholly of limestone. The two rocks were

separated by a well-defined, nearly vertical layer of shale, black in color, especially on the up-stream side, friable on the surface, but becoming harder a few feet below, particularly on the down-stream half of the foundation. The welding of the two main rocks, the gneiss and the limestone, with the shale, appeared to be quite complete at the depth of excavation finally reached. The surface of the limestone, from the point of junction toward the south, was nearly level for a distance of about 400 ft., until it reached well into the other side of the valley, where it rose gradually with the south slope. The limestone varied greatly in character throughout the extent uncovered. In places it was of sufficient compactness and water-tightness to answer for the foundations of the structure. In other places the general character was diversified by belts of varying width which were either full of eroded seams, through which water was found to flow freely when excavation was in progress, or masses of stone broken up by seams running in all directions, which were filled with mud. In addition, there were other well-defined belts, and all followed the general dip and strike of the rock, which, in the case of the dip, was nearly vertical, and of the strike, at right angles to the line of the dam following the valley. These last belts were of partly disintegrated, finely granulated limestone; were very well-defined and at the surface were easily removable with the pick; and grew harder and more compact with increased depth of excavation. These fissured, eroded and granular belts seem to form three distinct classes into which the bad features of this limestone bottom may be separated.

The different fissures developed many erosions in certain cases and were found at various points through the limestone stretch of the foundation, being larger and closer together as the junction with the gneiss was approached. These fissures, while well defined, were of varying widths, developing a line of erosions generally through very hard limestone.

As an illustration of the eroded seam, one case developed into a cave the location and existence of which were noted by tracing a narrow, horizontal seam in the rock near the surface, at about Station 7 + 70, 50 L., along the strike of the rock. This seam was in fairly solid rock, and clear water flowed from it. As the excavation along the line of this flow toward the up-stream side of the dam progressed, there was found a sharp downward dip, and the flowing stream soon required

FIG. 1.—MAY 27TH, 1897. STEPPING AT BOTTOM OF CORE-WALL.

FIG. 2.—MAY 26TH, 1896. LAYING FIRST STONE IN DEEP ROCK CUT.

for its management a subsidary pump. The seam enlarged into an erosion filled with sand, which, as it was followed, developed into a cave about 7 ft. x 9 ft. in section. This led under a heavy mass of solid rock to and beyond the up-stream line of the dam foundation. Connected with it were found, on the sides and in the roof, other erosions which were traceable nearly to the surface of the rock within the limits of the dam foundation, and which, on the up-stream side, outside face limits, in one case penetrated to the rock surface, where it showed as a narrow and somewhat prolonged fissure.*

While all the eroded fissures showed flows of water of varying degrees, several such were found which developed into strong springs, of which special care had to be taken. One, in particular, was found as the excavation in the rock deepened, limited and defined to an erosion in solid rock 6 or 7 ft. in diameter at about Station 6, near the up-stream side. The flow here was continuous and heavy, more than filling the 10-in. pipe which was at first placed to receive it, and afterward, as the spring hole was welled up in the foundation masonry, rising with this masonry and in the pipes which were at the same time placed in connection with the well, to a height of 90 ft. above its source before it was found advisable and expedient to attempt to fill it up and block it off. A particular and detailed account of all the operations connected with this spring will be found in the particular description of the treatment of the rock bottom.

As to the granular belts referred to, the excavation in them was carried down until the surface exposed was very compact. These surfaces were afterward tested for bearing power by means of an arrangement especially designed for that purpose and shown in Plate 20. Further allusions to these belts will also be found later.

In limiting the extent of the excavation vertically, the end aimed at was to reach rock sufficiently free from seams, and solid enough to afford all the bearing strength necessary to sustain the superimposed masonry and resulting pressure. The result involved a very large amount of deep rock excavation; the depth in one place being 50 ft. before satisfactory compact rock was found. It is not assumed that there may not be some tendency to upward pressure through some of the fissures which remained after the excavation was completed, but, as will be described later, every effort was made and every precaution

* A detailed description of this cave will be found further on.

taken to fill them, and it may be conceded that should upward pressure occur in some cases it must be reduced to the very small area presented by the mouth of the fissure in question to the bottom layer of the dam masonry, this area forming a very small proportion of the greater area against which upward pressure might be expected.

As to the possibility of percolation under the dam, that question would be more important if the rock bottom were exposed to the direct contact of the water in the reservoir, but it must be borne in mind that from the lowest point of the foundation of the main dam to the top of the back-filling above, there will be a compact filling of about 150 ft., in this particular case, which, while extreme, is not different, excepting in the great depth, from the condition which will obtain along the whole length of the masonry dam.

This question of possible percolation will be further considered in connection with the chapter on "Pumping."

It is an important and peculiar fact that, throughout the rock excavation of the whole foundation, in no case did the numerous test holes, drilled in the vicinity of seams and erosions, strike any openings in seams or rock which were not easily traceable by some continuous natural passage to the surface of the rock under preparation for the foundation. In other words, it may be fairly claimed that the existence of all open seams lying within 12 to 16 ft. of the dam in the various bad sections are traceable from natural indications at the surface. It is, therefore, to be assumed that all such seams were found and properly noted. A reference to the contour plan, Plate IV, will show that the variations and character of the limestone, and the necessary excavation, were much greater nearer its junction with the gneiss than at the south end, where, with the exception of a few eroded seams, the rock is uniformly hard and compact, and required but comparatively little excavation at the surface. It would seem that at some time the disturbance of the limestone formation must have been considerable; the greater part of it occurring near the point of junction. From developments indicated by a comparatively small amount of excavation in this part of the limestone foundation, and the fact that the general character of this bottom was naturally considered an important matter, it was deemed advisable, during the excavation of the first section of the bad rock, which lay at Station 8 + 50, to consult a specialist as to the general condition in which limestone

PLATE IV.
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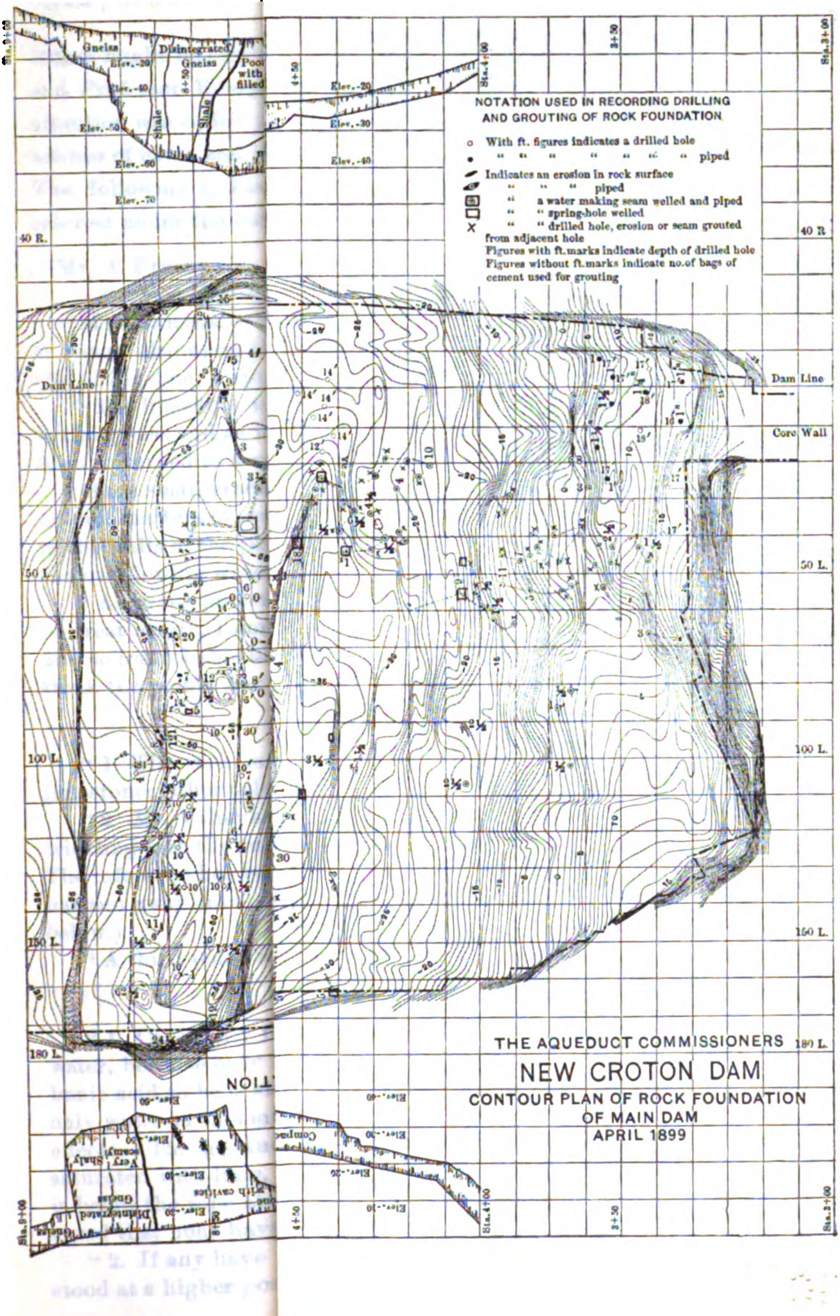
JANUARY, 1900.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



ledges might be expected to be found under the prevailing conditions, and Professor Kemp, of Columbia College, was consulted, and his attention was called particularly to the question of the probable existence of caves and similar openings under the general rock surface. The following is his report, which is introduced here as being of interest under the circumstances:

"NEW YORK, May 14th, 1896.

"MR. A. FTELEY,

"*Chief Engineer, Aqueduct Commission.*

"MY DEAR SIR,—In reply to your letter of the 12th, requesting me to report also upon the probable condition of the limestone under the site of the dam, I append the following to my report of two days ago.

"The limestone is undoubtedly more or less fissured precisely as is the gneiss and as is to be expected in regions where the rocks have been upturned to a vertical position.

"Such small cracks cannot of course be avoided and, I understand, are not matters of serious concern. They are the ones that now show in the walls of the pit and that let in the water in all probability from the overflowing water-soaked sands and gravels.

"As to the presence of large caves, several feet across or more, and of great length, I am of the opinion that their existence is improbable, and so improbable as not to give occasion for special treatment. I think the points *For* and *Against* them may be stated as follows:

"FOR.

"1. The rock is limestone, and caves are practically limited to limestone; other soluble rocks being rare with us.

"2. Mr. Value has been impressed with the fact that the water trickling into the sump has diminished as the pit has gone deeper. The inference has been suggested from this observation that the water has run away into some underground cavity. (See further under 5 below.)

"A third point is stated and discussed under 2 below.

"AGAINST.

"1. Caves only form above the level of the ground-water or well-water, because only freshly fallen rain is sufficiently charged with carbonic acid to be a strong enough solvent to be serious, and because only water in this situation flows rapidly enough to produce profound effects. The ground-water stands too still, and too soon becomes saturated with lime, to be effective. The present position of the rocks is below the zone at which caves could form, and it is practically assured that none have formed since they assumed this position.

"2. If any have formed, they must have done so when the rocks stood at a higher position and above the ground-water. We all believe

that this whole region was much elevated during the Glacial period, and it cannot be denied that conditions may have been favorable at that time. Some superficial decay apparently took place, as shown by the sandy streaks in the limestone, but after this time a strong stream must have flowed over these rocks to have availed to deposit the heavy burden of sands and gravel that rest upon them, and if any such cavity existed near the surface the probability is strong that it has been packed full of sand.

"3. The rocks stand vertically, and all underground drainage or circulation must tend to follow their bedding planes much more than to cross them. We would infer from this that any cavity would be long and narrow and not an easy thing to locate with a drill.

"4. No hollow sound, so far as I know, has been noted in the work in the pit, when picks, drills or the descending boxes from the cables have struck the bed-rock.

"5. In case the water has diminished, as observed by Mr. Value, I think it is due to the partial exhaustion of the neighboring gravels, for the weather has been dry and rainless for a long period, rather than to any cavities under the bottom of the pit. Such assumed cavities, being 50 to 100 ft. below the level of the Hudson River, and having stood for an indefinitely long time under wet gravels, must have been long since filled with water.

"6. All the experience, so far as I know, that has been gained in quarries in these limestone belts in New York and the neighboring parts of New England, has shown caves to be extremely rare. An assistant of mine has recently had occasion to visit every one of them, and he only met one small cave, which was at Hastings. Of course there may be others, and I am aware of the existence of a large cave near the Twin Lakes in the northwest corner of Connecticut, but, considering the abundance of the limestone areas, they are certainly rare.

"For these reasons I regard the probability of their existence under the site of the dam as remote.

"Very respectfully yours,

"(Signed) J. F. KEMP."

In order to explore this limestone bottom more completely, it was found advisable to drill a few test holes of considerable depth. These were accordingly undertaken at certain points in the bottom where it was nearly ready for the masonry, and their location, direction and depth, are shown on the contour plan.

An extended account and description of the excavated rock bottom, for the main dam foundation, referring particularly to the limestone bottom and to the various changes and characteristics shown by this rock, can be found in a report made by the author to the Chief Engi-

FIG. 1.—MAY 26TH, 1896. MAIN DAM EXCAVATION, LOOKING SOUTHWEST.

FIG. 2.—MAY 19TH, 1896. MAIN DAM EXCAVATION. DEEP ROCK CUT, LOOKING WEST.

neer of the Aqueduct Commission, in which, for the purpose of a record, all the facts are noted in considerable detail. Constant reference is had to Plate IV, which is a plan in contour of the finished rock bottom between Station 3 + 20 and Station 9, and which shows contour elevations at intervals of 1 ft. On it are shown also, by means of cross lines, the limits of the different belts in the rock, and the heavy dotted line shows the neat lines of the dam foundation masonry.

The shale seam, separating the gneiss and limestone, lies at Station 8 + 80 ±, varied in width at different points, but grew narrower and more solid as the depth of excavation increased. Fig. 2, Plate III, shows the shale in the face of the cut and in the trench bottom on the left, where, however, the excavation shown is unfinished. The view is taken looking up stream.

The next of the series of belts into which the foundation may be divided, and which are in a measure indicated by the profiles of the finished bottom shown in Plate IV, and in Figs. 4, 5 and 6, extends from Station 8 + 20 to Station 8 + 70. Its character, when the excavation was about completed, is shown in Fig. 2, Plate III, and Figs. 1 and 2, Plate V. The contour plan, Plate IV, shows the number and depth of the search holes drilled in preparing the bottom for masonry. In this case they followed principally the lines of the erosions in the hard rock bottom.

Next beyond lies a section, showing also in Fig. 1, Plate V, between Stations 7 + 80 and 8 + 20, and forming a solid ridge of hard, compact limestone, requiring but little excavation, comparatively.

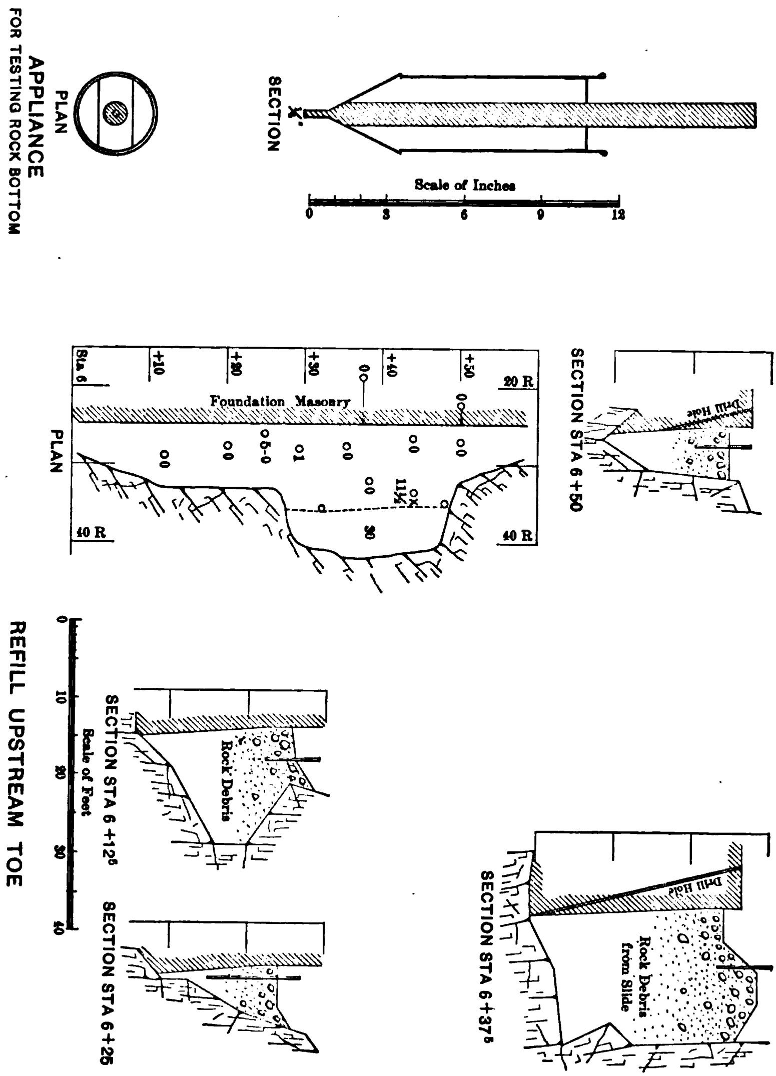
A well-defined narrow seam along Station 7 + 70 is illustrated by Fig. 1, Plate VI, showing its down-stream end. Its up-stream end developed upon excavation into the cave previously referred to. Fig. 2, Plate VI, shows the cave opening at Station 7 + 70, 25 L. The pump and suction hose in use are also shown. This suction hose and another line, of the same size, were built in the masonry when the tunnel was filled up, in order that the necessary drainage from a sump hole, placed outside of the upper line of the dam in the lowest point of the tunnel reached, might be maintained. The length of the cave excavated and filled as tunnel was about 30 ft. The floor is of very hard and solid rock; holes traced 16 ft. deep found no openings below. The masses of rock on the top and sides are all solid, showing few or no open seams, except that the seam between the cave floor and the right side wall may have had

an open connection with the low point in the excavation at Station 7 + 35, 15 R., where, in grouting, later, there were some indications of an open passage between the points in question. As this was indicated by the pump from the sump at Station 7 + 62, 25 R., throwing out grout which was being pumped in at the point noted, it is not at all certain that the line of communication between the two did not lie mostly outside the dam foundation.

To facilitate the work of filling up the cave, a small shaft was sunk at Station 7 + 73, 23 R., to strike one of the subsidiary caves found on the upper line of the dam. Two other and smaller eroded chambers, leading into the roof of the main cave, were also found. These spaces were all filled, within the lines of the dam, with rubble masonry, or, in the case of the two small caves shown in plan on the contour plan at Station 7 + 70, and Station 7 + 78, with well-packed small stone, placed from above through openings made for the purpose in the overhead rock, and then filled with grout.

The sections and sketches opposite Station 7 + 70, Fig. 7, show the location and extent of these various caves, and it may be noted that in filling up the main cave and the branch cave, *i. e.*, entering above the main cave roof from above the up-stream line of the foundation masonry, care was taken to build this masonry filling 4 ft. beyond the limiting line. The large cave beyond the line, was found to be about half full of sand and gravel when it was reached from the tunnel and the small shaft sunk in its roof. It is evident that originally the space had been solidly filled, but that the cleaning of the tunnel, and the pumping of the heavy water flow had caused the partial emptying of the filled space beyond. It was refilled later, after the masonry filling had been built, by washing gravel down from the slope outside through openings made in the surface of the rock for that purpose.

Between Stations 7 + 30 and 7 + 60 is shown a narrow well-defined seam of hard rock with many erosions connected and extending to the deeper holes excavated at the ends. Fig. 1, Plate VI, shows on the left the deep excavation on the down-stream end. Beyond this seam lies a compact seam of friable limestone about 10 ft. wide. It is shown in Fig. 1, Plate VI, on the extreme left, where the ladder is resting against it when the excavation was completed. It was tested for bearing strength by an apparatus shown in Fig. 10. This apparatus consists of a cylinder to be loaded with shot necessary to produce



APPLIANCE
FOR TESTING ROCK BOTTOM
PLAN
SECTION
X
Scale of Inches
0 3 6 9 12
Sta 6
+10
+20
+30
+40
+50
20 R
40 R
40 R
Foundation Masonry
PLAN
SECTION STA 6+50
Drill Hole
SECTION STA 6+37⁵
Drill Hole
Rock Debris from Slide
SECTION STA 6+12⁵
Rock Debris
SECTION STA 6+25
Scale of Feet
0 10 20 30 40
REFILL UPSTREAM TOE

FIG. 10.

the required pressure upon its bearing point, a circle ¼ in. in diameter. This was applied carefully and repeatedly to the surface in question at different points, and the results indicated that the bearing power of the surface was ample up to the limit of the test, which was 250 lbs. to the square inch.

A second but much narrower section of the soft granular rock just mentioned lies a short distance beyond. This shows on Plate VI, as from 3 to 5 ft. in width, and beyond this is a wide stretch of bottom shown on the plan as being composed of alternate strata of "soft and granular limestone and hard eroded limestone." In this the most extensive and deepest excavation occurred. Figs. 1 and 2, Plate VII, which are views looking up stream, illustrate the character of the excavation. Fig. 2, Plate VII, shows the deepest point reached, *i. e.*, Elevation —80.4 below datum. Figs. 1 and 2, Plate VIII, show the character of the rock in the same vicinity more in detail, and the location of some of the erosions through which excavation was made. In Fig. 2, Plate VII, is shown the spring hole at Station 5 + 93, with the iron pipe in use to convey the flow.

The sloping bed of rock shown in Fig. 2, Plate VII, extends quite across the dam; and beyond it, to Stations 5 + 40, lies a hard bottom, reasonably free from seams and erosions, and calling for but little excavation. Between Stations 5 + 30 and 5 + 40 occurs a deep, well-defined seam, or series of erosions, shown at the upper end in Fig. 1, Plate IX. This photograph shows also the general character of the rock bottom on both sides of this seam, and Fig. 2, Plate IX, and Fig. 1, Plate X, show it still further to the south, and to the junction of the main dam with the core-wall. This part of the bottom calls for little comment, although a seam showing at Station 4 + 60, and extending partly across the foundation, is shown nearly ready for masonry in Fig. 1, Plate X.

In addition to the search holes, which were numerous, and were from 12 to 14 ft. in depth, it was thought advisable to drill a few test holes of considerable depth. They were accordingly undertaken at certain points in the bottom where it was nearly ready for masonry, and their location, direction and depth are shown on the contour plan. The first, or No. 1, was located at Station 8 + 68, 103 L. Length drilled, 48.4 ft.; direction and dip indicated by the black arrow ending at 8 + 52, 100 L. Hole No. 2 was located at Station

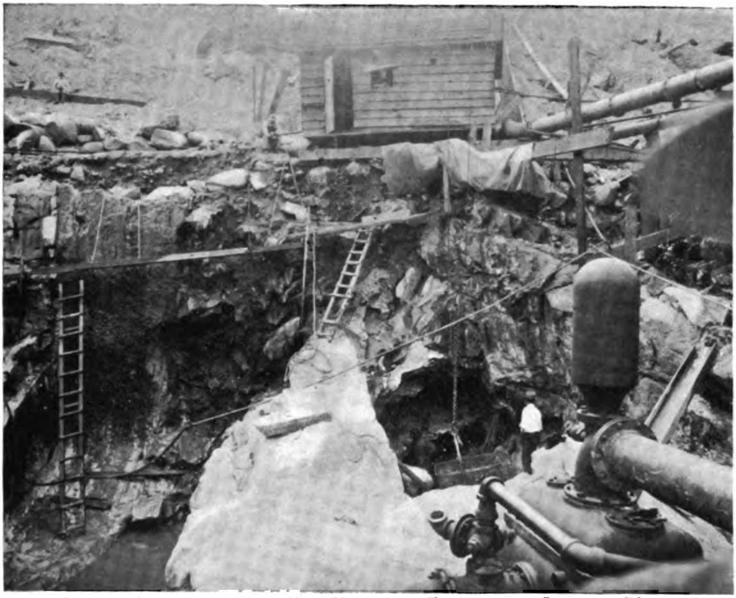


FIG. 1.—SEPTEMBER 1ST, 1896. MAIN DAM EXCAVATION, LOOKING WEST.

FIG. 2.—SEPTEMBER 12TH, 1896. CAVE AT STATION 7 + 70, 10 L.

7 + 76, 84 L. Length drilled, 100.6 ft.; direction and depth indicated by arrow ending at Station 7 + 16, 86 L. Hole No. 3 located at Station 7 + 52, 5 R. Length drilled, 100.6 ft.; direction and dip indicated by arrow ending at Station 7 + 14, 50 L. Hole No. 4 is located at Station 7 + 50.5, 75.8 L., and was drilled vertically 55.6 ft. deep.

The three inclined holes were drilled in May, June and July, 1897, while the sump hole at Station 8 + 50, 10 L., the bottom of which was at Elevation —67, was in use. They were inclined in order to cross the vertical seams, and to better the chance of finding large erosions or caves. The results did not seem to indicate anything more extensive in the way of erosions than had already been found by the excavation for the first bottom formed between Stations 8 + 50 and 8 + 70, and it was some time later in the season that the excavation in its regular progress developed the location of the large cave to which attention has been called.

The profiles, Figs. 4, 5 and 6, show the excavation necessary to get below the loose and open seams, which in places was considerable, as the seams separated the rock into heavy solid masses which they bounded on all sides. The bottom reached was solid, compact and tight. At the date of writing, a narrow strip of this bottom, close to the river wall foundation, remains to be excavated. This was left, when the great bulk of the work was completed, as, at that time, it furnished the foundation for a trestle then in use. This remaining strip is about 20 ft. wide, and, with the section of the overflow bottom now under process of work, completes all that is incomplete in the foundation excavation of the whole dam.

Grouting and General Treatment of the Rock Foundation.

Upon the completion of the rock excavation of any particular section of the bottom, which work included in many places a prolonged and tedious barring out and cleaning up of shaky or loose pieces of rock, the bottom was washed down and thoroughly cleaned by streams of water under a heavy head; and operations were then begun to clean out all erosions and open seams showing at the surface, and to trace them out as thoroughly and as far as possible by drilling numerous holes of varying depths in their vicinity.

All erosions and open seams were, as a rule, filled with Portland

cement grout mixed with fine sharp sand of 1 to 1 or 2 to 1 mixture according as the grout was pumped or poured. In the case of the "cave," rubble masonry in mortar was used as filling in the large opening, while in some smaller erosions the spaces were thoroughly packed with small stones before the grout was poured in, and, in one or two exceptional cases, American cement was used for the grouting.

The drilling of holes in the rock bottom for grouting and searching purposes was begun as soon as the first section of bottom was excavated. Air drills were used, and were fed from the pipes used for drills at work on the general excavation. In all, about 1 700 lin. ft. of holes were drilled. They were about 2¼ ins. in diameter at the rock surface, decreasing somewhat as the depth increased. These holes were of all depths up to about 16 ft., according to circumstances. Whenever it was found impracticable or inadvisable to pour or pump grout into holes or erosions before the adjacent masonry work was started, vertical pipes, generally 2 ins. in diameter, were placed in them and were then built around with the masonry up to such height as was necessary. In case holes or erosions showed a flow of water, such openings were also provided with pipes placed at proper inclinations to lead to some drain center or sump hole near by. In most cases the erosions, and in a majority of cases the drilled holes, were piped, as it was found more convenient and practicable to pump grout under heavy pressure into openings thus prepared and sealed and covered with masonry carried up to some convenient height. Old steam-piping was generally used for this purpose, and, while the diameter was generally 2 ins., other and larger sizes were sometimes provided, and in the bottom hard tile piping was at times used to carry water flows. In the case of the heavy spring at Station 5 + 95 large-sized, galvanized, riveted pipe was used to connect with the spring as the foundation masonry was built up, and at the cave at Station 7 + 70 it was found necessary to build into the filling masonry two 10-in. galvanized-iron suction pipes which were afterward filled by pumping grout.

Whenever grout was pumped a No. 2 Douglass deck pump was used. The grout was mixed by hand in boxes made for the purpose. The suction and delivery hose were each 3 ins. interior diameter, coupling to 2-in. holes at the pumps and also to a 2-in. nozzle, at the outer end of the discharge hose, made of a short piece of steam pipe. When

FIG. 1.—NOVEMBER 20TH, 1896. MAIN DAM EXCAVATION. ROCK FACE, UP-STREAM SIDE.

FIG. 2.—FEBRUARY 26TH, 1897. MAIN DAM ROCK EXCAVATION AND MASONRY.

in use, this nozzle was either set well into the pipe leading to the channel to be grouted and carefully packed with waste and old bagging, or, in some cases, was coupled to the grout pipes by the use of screw threads and couplings, cut and furnished for that purpose. From four to six men were used at the pump handles, according to the resistance experienced in forcing the grout, and the pressure developed was sufficient in cases to burst the hose while still forcing appreciable quantities of grout to flow. As a preliminary to grouting any hole or section, care was always taken to flush the pipe and passage very thoroughly with water under a heavy head; a system of pipes leading from the Old Aqueduct, which was at an elevation of from 200 to 250 ft. above the main dam foundation, furnishing all the facilities for this purpose.

The contour plan, Plate IV, shows the position and extent of the various fissure erosions, cave and springs treated together with the location and depths of all holes drilled in the process of tracing out; and also figures showing the number of bags of cement used in grouting at various places.

The first section grouted was between Stations 8 + 20 and 8 + 70. The search holes drilled had in many cases established a connection with the lines of erosions showing in the bottoms, and most of the holes were filled by the flow from holes adjoining. In this section 701 bags of Portland cement (175 bbls.) were used, mixed with sand (2 to 1). All erosions of any size were filled with small stones before being treated with grout.

The next section treated was along line 7 + 70, and included the cave which, after being cleared of gravel and having control of the water flow, gained by means of a 10-in. double Worthington pump (1 500 000 galls. per 24 hours), was filled with rubble masonry laid in Portland cement mortar. The sump-hole was established outside the line of the dam at about 27 R., and its location has been shown at the ends of the two suction pipes which, as stated, were left in and built around solid with masonry. The larger pipe was the one used in connection with the 10-in. pump. The second pipe was placed about 2 ft. higher than the other and was provided in order that drainage might be maintained in case the lower pipe should clog with sand or gravel during the filling up of the cave. A third pipe 3 ins. in diameter, reaching partly through the cave, was laid on the bottom below the

lower suction pipe and reached to the sump-hole. It was used to take slight flows from the walls and floor to the sump, and was also built in when the cave was filled. During this process the lower suction became clogged and the pump was connected temporarily with the upper pipe, while a stream of compressed air was blown through the 3-in. pipe. This freed the outer end of the lower suction which was again put in use, and no further trouble was had with it.

Fig. 7 shows cross-sections of the cave at various points. The section at 00 D. L. shows on the left the connection with the long oval-shaped pocket shown on the plan at Station 7 + 78. It also shows on the right a connection with a smaller pocket at Station 7 + 70. These pockets were comparatively deep, and plainly showed erosion between the solid stratum forming the cave roof and the somewhat softer stone on each side. The section at 125 R. shows the average cave section inside the masonry lines. At 25 R., just outside the up-stream line, the cave abruptly enlarges, reaching nearly to the rock surface, while at 37½ R., the section is somewhat smaller apparently, although it was not free enough of gravel and sand to show that clearly.

As the cave was cleared out it was heavily timbered in the roof for the protection of the workmen from the possible fall of detached pieces of rock, and, when 27 R. was reached, a timber bulkhead about 4 ft. high was built to retain the gravel slope lying in the fissure beyond. It was also of use in forming the outer wall of the sump-hole which was located on the extreme right of the cave where the roof was low. A shaft 4 to 6 ft. square and about 6 ft. deep, between Stations 7 + 69 and 7 + 75, was sunk to reach the roof of the enlarged cave section which, at this point, ran back about 10 ft. from the outer neat line of the dam, re-entering for that distance over the roof of the cave proper and near the rock surface. The masonry filling began close to the bulkhead line, 27 R., and was carried up on that line vertically to the surface through the 4 x 6-ft. shaft which was excavated for that purpose. As this filling progressed in the cave, working toward the center of the dam, the timber was gradually removed with the exception of two 8 x 12-in. range timbers which extended throughout its whole length and which were built in. A small shaft was sunk into the roof of the oval-shaped pocket on the left, at Station 7 + 78, and when the cave masonry below had been built to the general roof level this pocket was filled with small stones and then grouted, taking

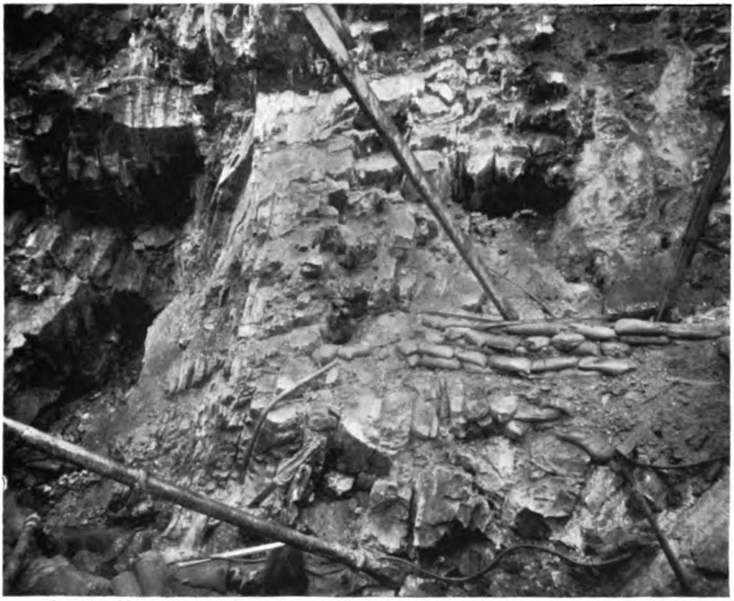


FIG. 1.—MARCH 2D, 1897. MAIN DAM ROCK EXCAVATION. UP-STREAM FACE.

FIG. 2.—FEBRUARY 17TH, 1897. ROCK BOTTOM AND EROSIONS.

forty-eight bags of cement (2 to 1 mixture). The smaller pocket in the right was packed with stones from below and grouted through an inclined drilled hole 12 ft. deep, taking eight bags of cement (2 to 1 mixture). Other inclined holes were drilled in this vicinity—12 to 18 ft. in depth—in a search for further cavities.

To the pump ends of the suction pipes as they were built in, reducers and 2-in. iron pipes were finally attached, and the water from the sump-hole outside was allowed for a long time to flow through and was conducted to a temporary sump-hole near the center of the work, while the masonry was gradually built up. As these pipes were raised higher, this flow finally stopped, as the back-filling on the up-stream side then in place was not sufficient to prevent the flow from finding an outlet in the up-stream sump to the south. These suction pipes were grouted when the masonry and connecting pipes had been raised about 40 ft., 72 bags of Rosendale cement (1 to 1 mixture) were poured into the lower and longer pipe, filling it, then 42 bags of Rosendale cement (1 to 1) were partly poured and partly pumped into the upper and shorter pipe. The pumped material was forced up through the back filling on the up-stream side and this caused a temporary stopping of the experiment. Some time later (about a year), 11 bags more were pumped in, and the hole was blocked, no sign of this grout showing this time in the up-stream back filling, which, in the meantime, had been carried up much higher. In the narrow, eroded seam lying along the line from Station 7 + 32, up stream, to Station 7 + 58 down stream, 300 bags (75 bbls.) of cement were used, the grouting showing at least connections between adjacent erosions and search holes as the seam was pumped full.

Beyond, between Stations 6 + 80 and 7 + 10, and partly including a bottom which was hard and solid, but full of open seams and erosions, and distinguished by some solid masses which rose above the general surface, a great many pipes were used, and a large quantity of grout was pumped in.

The next bottom section, covering the lowest point reached for a foundation, was drilled and treated as usual, but the extreme low bottom on the up-stream side took but little grout except along the line of seams from 20 R. to 30 L. near Stations 6 + 50.

On the down-stream side will be noted some lines of erosions into which considerable grout was pumped.

But little grout was used beyond this bottom until the eroded seam between Stations 5 + 30 and 5 + 40 was reached, as the spring hole at Stations 5 + 93 was not treated by grouting.

The erosions along the 5 + 40 line were drained at a sump-hole at 39 L. during the excavation, while the bottom masonry was being laid and the drilled holes and erosions were being piped. A well was therefore gradually built up at this point, reaching a depth or height of about 20 ft. before the grouting work was started. The holes in the seam between the well and the up-stream side were grouted by pumping before the well had reached this height, as there was no connection between them, the water in the well coming wholly from the other direction. When the well was ready the drainage pump was taken out and the drainage was maintained by a pump attached to the 5 in. pipe shown at 165 L. This pipe was just outside the down-stream toe line of the dam, and had been placed and used for a drainage well while the rock excavation in its vicinity was being made.

The main well hole, as it was built up, in places had its down-stream face built of stones laid dry, in order that seams in the adjoining rock might not be shut off from the grout later, as well as to allow free passage of the water to the suction pipes. A 2-in. pipe was also built into this well, reaching to its lowest point and connecting there with seams in the rock.

The well was filled with 80 bags of Portland cement (1 to 1 mixture) poured in, and it was evident from the water which was forced from pipes nearby, notably at 54 L., that the grout was reaching the seams and passages in that direction. As the grout was poured the well was gradually filled with small stones collected for that purpose, After no more grout could be poured 4 bags of cement (1 to 1 mixture) were pumped into the pipe placed in its corner. The grout pump was then tried in each pipe in turn working toward the down-stream side. The grout was forced gradually into the 5-in. pipe, the pump at which was stopped when the grout trace became marked. This pipe was filled, so that the water flow ceased through it, by pumping at the spring holes near by at 150 and 158 L. Later, after the pump was disconnected, it was completely filled by pouring 5 bags of cement (1 to 1 mixture) into it at the top. By this time, the water which had flowed along this seam was blocked off entirely and had forced its way up to

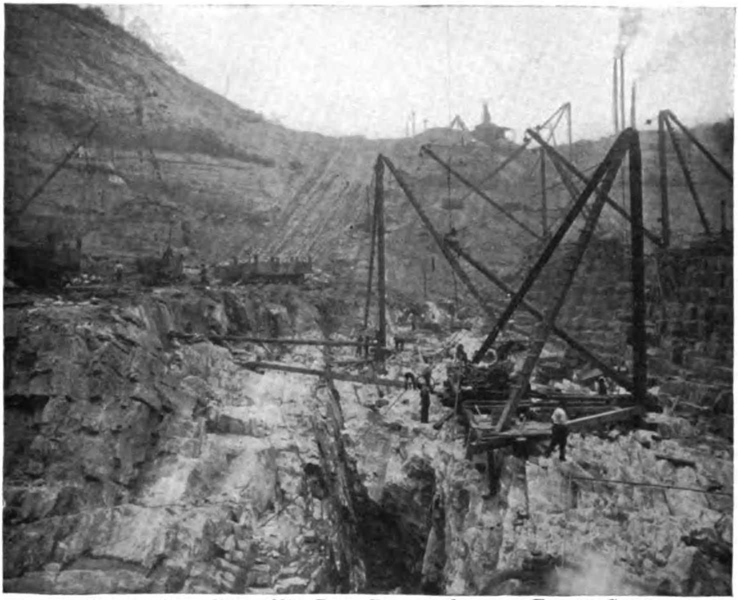


FIG. 1.—JUNE 27TH, 1897. ROCK BOTTOM, SHOWING ERODED SEAM.

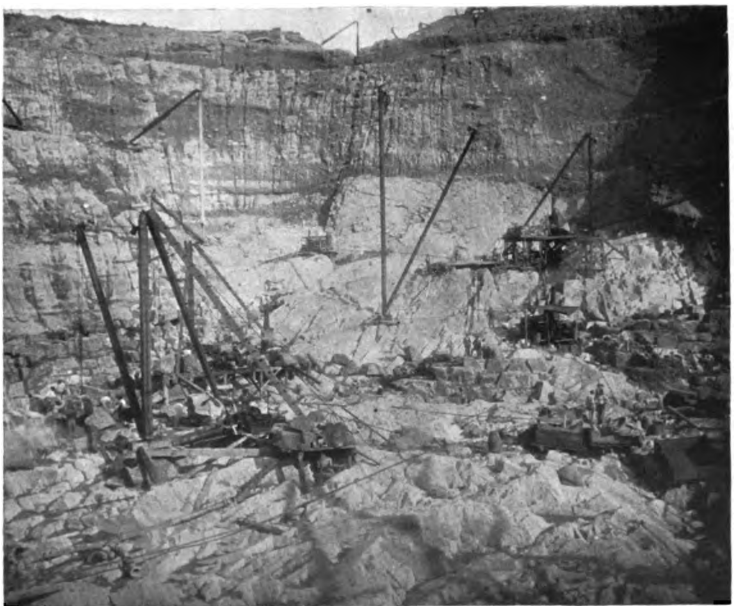


FIG. 2.—AUGUST 12TH, 1897. ROCK EXCAVATION, LOOKING SOUTH FROM STATION 6 + 00, 60 L.

the surface of the down-stream gravel slope at an elevation considerably above the top of the 5-in. pipe.

Beyond this seam, little grouting was found necessary until Station 4 + 60 was reached, where the seam developed in the course of the rock excavation showed some traces of erosion on the solid vertical face left in on the north side. The nearly horizontal open seam reaching under this vertical face showed a few springs, and at two places, 43 and 126 L., 18 and 30 bags of cement (1 to 1 mixture), respectively, were pumped in, and smaller quantities at other places. The spring at 43 L. was filled and "X"d at three points between 25 and 31 L. The grouting nearer the down-stream side gradually drove the water outside the masonry limits. Six holes, 12 to 14 ft. in depth, were drilled on the line of this seam near the up-stream side, but further traces of it were not found. A number of pipes were placed between 18 and 45 L., Station 4 + 10 to Station 4 + 40, above and along very narrow but somewhat open seams in masses of solid white rock, and, as the bottom masonry was laid, these pipes were connected by covering the seams with small spawls laid dry. The pumping afterward done indicated free flowing between the pipes, and a considerable portion of the grout must have been used to fill channels thus provided.

The same remarks apply to the piped seams from 40 to 60 L., Station 3 + 50 to Station 4 + 10, where the open seams were in most cases so fine that they could have taken but little of the grout pumped. A wider seam at Station 3 + 75, 0 to 30 L., was piped and took an appreciable quantity, as is shown on the plan. There were no signs of erosion there.

The drilled holes, 16 to 18 ft. deep, between Stations 3 + 30 and 3 + 60, practically took no grout. Neat cement was pumped into them, and the amount taken was only about enough to fill the holes.

They were drilled, as the rock, though compact and fairly solid, was full of short heads and tight seams, and it was thought best to make sure that the seams were no looser below.

The Spring at Station 5 + 93.

It was at first proposed to grout this spring, as well as the others, but circumstances led finally to a different course of treatment in this case.

The flow from this spring was heavy. When first uncovered it was curbed with a bag dam and piped as shown in Fig. 2, Plate VII. The

pipe was 8 ins. in diameter, and the flow was sufficient to back up against the pipe at the entrance. The inclination of the pipe, however, helped the flow, and at its lower end the pipe was about half full. It was unfortunate that no gauging of this flow was ever made, but, with many other springs and flows about, it was overlooked.

This flow was carried through this pipe for some months until the masonry work had reached the spring level, when it was taken by a 12-in. iron pipe laid in the masonry to the sump-hole by this time established near by, outside the up-stream face which had been carried up to about 30 ft. above the lowest point of the rock bottom. Later, another 12-in. pipe was laid from a slightly higher elevation through the masonry outside the face of the dam. This pipe was continued with a 90° elbow, and, after plugging the lower pipe, short vertical lengths of pipe were added to it by which the point of discharge was gradually raised as the masonry forming the well above the spring was carried up ahead of the discharge pipe. This arrangement also allowed the back-filling against the up-stream face to be carried on conveniently without impeding the flow of the spring or filling it with earthy material. The head of the spring was reached at about Elevation + 20, when the flow which had been gradually diminishing ceased. This was about 83 ft. above the rock bottom of the spring hole and 74 ft. above the outlet pipe which by this time had served its purpose for 15 months while the masonry was building. The section in Fig. 9 shows the above-described features, as well as the partial location and trend of the open fissure leading from the spring hole. This fissure showed at the well hole a considerable section; its direction was down stream with a downward horizontal dip. Viewed from above the well hole, the bottom was full of well-washed gravel stones of comparatively small size. As the passage was always full of water flowing swiftly, it could not well be explored, but the line of drilled holes shown on the contour plan from 7 to 17 ft. in depth was used to locate its position and direction as far as practicable. Five of these holes reached the fissure or connecting seams and were piped with 2-in. pipes until the head of the spring was reached. The additional sections shown in Fig. 9 are from the results of these borings. When the well had been built to Elevation — 2, it was decided to seal it, as to carry it higher involved unnecessary complications with the masonry work. It was 4 ft. square. A 3-in. iron pipe

was placed in one corner and reached nearly to the bottom. A large flat stone was lowered, and by tag and guide ropes so placed as to partly cover and shield the 12-in. pipe opening. Its position was directed by sounding with plummet and wire line, and two trials only were necessary to place the stone properly. The well was then filled up to Elevation — 47, about 8 ft. above the outlet pipe, with clean spawls, the larger sizes being placed on the bottom and gradually diminishing to the size of concrete material at the top. These stones were lowered to place in a box built to fit the well with a bottom which could be tripped open. The same box was used to continue the filling with concrete, Portland cement in the proportion of 4 gravel, 2 sand and 1 cement being used. The dump box had a capacity of 18 cu. ft. The first batch of concrete was mixed dry and was placed in the box on a sheet of canvas which covered the bottom and sides of the interior, and formed a tight bottom for the concrete mixture when it was dumped. The dumping of the concrete was continued diligently until it had risen about 13 ft. This work had not in any way disturbed the flow through the outlet pipe which showed no discoloration due to cement or gravel, and examination of the 3-in. pipe in the well corner indicated that the sealing was complete, as the displaced water in the well was overflowing at the top, while the water in the 3-in. pipe remained stationary. This was further shown the next day when the well was bailed out nearly dry to facilitate its further filling with concrete. When filled the masonry work above and around the well was resumed, but the 3-in. pipe was carried up with the others until the head of the spring was reached.

The four 2-in. pipes furthest away from the well hole took water freely. They were about 90 ft. long. The water from the spring rose in the line of pipes to Elevation 20 ±, about 10 ft. below their tops. It was a question whether grout could be poured successfully through so much water, and in the first pipe tried, 17 L., the grout clogged half-way down the pipe, owing, apparently, to some roughness due to carelessness in joining the sections of the pipe. This was washed out by a flow of water pumped through a $\frac{1}{2}$-in. pipe, and the pipe was cleared.

It was then suggested that an effort be made to fill this erosion with plastic clay by driving through the pipes. This suggestion came from the contractors, who had used clay to fill cavities under some-

what different conditions. Arrangements were made to try this method, and a small pile-driver with a 2 000 lb. hammer was set up over the 2-in. hole at 32 L.

A piece of 3-in. steam-pipe, about 6 ft. long, was used at first as a receiving cylinder, and it was connected to the 2-in. pipe which projected above the masonry about 1 ft., with a strong reducing coupling. The follower used was a $\frac{7}{8}$-in. steel rod, of the length of the cylinder, turned to fit closely its full length. It was welded at its upper end to a lengthening rod, slightly smaller in diameter, which was fastened at its upper end to a wooden cross-head, designed to work in the guides of the pile-driver and to take the blow of the hammer.

Blue clay of good quality was used, and was made plastic with water and a thorough working and pounding into boxes 10 ins. deep and of a size to hold 10 cu. ft. These boxes were limited in dimensions simply for convenience in keeping record of the clay used. The clay was then cut into "sausages" 10 ins. long and $2\frac{3}{4}$ ins. in diameter by shovels which had been bent and sharpened for the purpose. As often as the shovel was used to make a cut it was dipped in a pail of water in order to lubricate its surface and free itself for the next cut. The sausages were passed to the pile-driver in boxes holding 50 lbs., and the amounts of clay thus determined were used for the purposes of a record, and later, were reduced to cubic yards.

The first hole to be tried was one which took water very slowly. On starting the clay driving, the machine worked very satisfactorily, but the clay drove hard and only $3\frac{1}{2}$ cu. ft. were driven in all; probably not much more than enough to fill the pipe, which was about 90 ft. long, and the fissure at its bottom. The next hole tried was at 17 L. It was also a slow water hole. Into this 332 lbs. of clay were driven, with the 3-in. cylinder, everything working hard, and an 18-in. drop of the hammer being necessary. It was then decided to change the 3-in. cylinder for a 2-in., with, of course, a correspondingly smaller piston. This piston proved to be somewhat loose in its fit, but the driving continued to be very hard and only 37 lbs. of clay in addition, were put into this hole.

The driving was then resumed at the hole at 10 L., the 3-in. cylinder being again used while a new piston was being fitted for the 2-in. cylinder. The hole took water very freely, and 200 lbs. of clay were driven easily, using a 1 ft. drop. The next 200 lbs. went harder and

required a 3-ft. drop before it was all in. A change was then made to the 2-in. cylinder and piston, but the difficulty of driving seemed to increase although a drop of 4 ft. was given the hammer. With this, only 58 lbs. more were driven in, and then the pipe split between the coupling at the foot of the cylinder and the masonry. After repairing this break driving was resumed, with a 4-ft. drop of the hammer, for 2 hours and very little clay was gotten in, when it was noticed that everything worked more easily and a drop of only 2 ft. was necessary. In the course of another 2 hours less drop was used and cracks began to be noticed in the surface of the masonry radiating from the pipe in use. Further effort confirmed this, and, on the next day, observations with level and transit, during a short period of driving, in which 50 lbs. were easily put into the hole, showed a distinct and appreciable rise in the masonry surface. In all, about 150 lbs. were driven in this pipe after the split had been repaired.

Leaving the upheaved masonry to be investigated later, the driving was transferred to the 3-in. pipe which had been built into the well at Station 5 + 93 as it was filled up. There was no question about a free flow through this pipe and a plumb-bob dropped in readily found bottom in the well below the known elevation of the bottom of the pipe. However, in view of the difficulty experienced in driving clay through the 2-in. pipes already tried, it was thought advisable to make a test to determine to what extent, if at all, skin friction interfered with the passage of the clay.

The 3-in. cylinder was therefore connected by a quarter turn with 97 ft. of 3-in. pipe resting on the masonry surface. At the further end another quarter turn and a 2-ft. length of pipe gave opportunity to fit a poppet valve, which was set at 34 lbs. per square inch. The pipe was then filled with water, and clay was gradually forced in from the cylinder end. It was found to require no force beyond the weight of the hammer without impact and the water was forced through the poppet valve as fast as the clay was pushed in at the outer end. Later, 28½ ft. of 2-in. pipe and 20 ft. of 1½-in. pipe were joined to the 3-in. pipe and the valve fixed at the outer end. Under these circumstances it took a drop of about 14 ins. to force clay to the extreme end and through the valve. An examination of the clay as it was forced from the ends of the various sizes of pipes showed clearly that, under even a very slight compression, the water is driven to the clay surface next

the interior surface of the pipes and acts as an efficient lubricator, the skin friction amounting to practically nothing.

Driving was then resumed at the 3-in. pipe; 3 250 lbs. of clay were forced in, only the weight of the hammer being needed on the first day.

Appended is an abstract from the log of the clay driving, following the work above noted:

"Tuesday, December 27th, 1898.—4 500 lbs. driven in 3-in. pipe; hammer only used; no impact to force clay. No evidence of clay in 8-in. pipe outside of masonry, though distinct tremor in water was noticed during driving. Water in 8-in. pipe remained at constant height, about 6 ft. from top of pipe. Elevation, 18.4.

"Wednesday, December 28th, 1898.—4 400 lbs. put in. About 8.45 A. M. water in 8-in. pipe had risen to within 20 ins. of top, and rose 5 ins. at each stroke of piston, falling back to old level after stroke. About 10 A. M. water began to flow over edge of pipe at each charge, but settled back below edge after stroke. About 12.00 M. water ceased to settle back. About 3.00 P. M. began to run in a small stream after stroke had been made. Plumbed pipe, but found no clay in it. Distance, measured from top down, 81.04 ft.

"Thursday, December 29th, 1898.—A small stream of water was flowing from 8-in. pipe when work started. 5 000 lbs. driven to-day. Plumbed 8-in. pipe as follows:

9.00 A. M.	Distance	81.04 ft.	—No clay in pipe.
2.50 P. M.	"	78.62 "	—2.42 ft. of clay in pipe.
3.50 "	"	78.08 "	—Rise of 0.54 ft. of clay with 750 lbs. put in between 2.50 and 3.50 P. M.

"At 2.50 P. M. water flowing from pipe was strongly colored with clay, which gradually cleared, and about 4.00 P. M. no trace of color could be detected in water flow.

"Friday, December 30th, 1898.—Clay driven 5 200 lbs. Plumbed 8-in. pipe as follows:

9.20 A. M.	Distance	77.28 ft.	—Rise 0.80 ft.	1 300	lbs. clay.
11.20 "	"	76.28 "	— " 1.00 "	1 250	"
2.20 P. M.	"	71.94 "	— " 4.32 "	1 850	"
4.00 "	"	70.88 "	— " 1.06 "	800	"

"At 11.20 A. M. water was running clear from 8-in. pipe and reduced in amount over yesterday, with very slight acceleration in flow, when charge was driven.

"Saturday, December 31st, 1898.—3 750 lbs. driven. Plumbed 8-in. pipe as follows:

9.20 A. M.	Distance	68.85 ft.	—Rise 2.03 ft.	2 150	lbs. clay.
10.20 "	"	68.15 "	— " 0.70 "	1 000	"
11.20 "	"	67.80 "	— " 0.35 "	1 000	"

"Water flowing from 8-in. pipe is clear and reduced in amount over yesterday, shows very slightly the effect of each charge. Total clay driven through 3-in. pipe to date, 26 100 lbs.

"Wednesday, January 4th, 1899.—4 000 lbs. clay driven. Plumbed 8-in. pipe as follows:

12.50 P. M.	Distance	67.70 ft.	1 000 lbs. clay.
2.00 "	"	67.60 "	1 000 "
3.10 "	"	67.55 "	1 000 "
4.45 "	"	67.50 "	1 000 "

"Water flowing from 8-in. pipe has decreased slightly since December 31st; runs clear and shows, very slightly, effect of driving charges. The weight of hammer only, continues to be required to drive clay down.

"Thursday, January 5th, 1899.—5 300 lbs. of clay driven. Weight of hammer only, required; measurements taken on 8-in. pipe as follows:

9.55 A. M.	Distance	67.30 ft.	1 000 lbs. clay.
11.25 "	"	67.22 "	1 000 "
1.05 P. M.	"	67.15 "	1 000 "
2.15 "	"	67.07 "	1 000 "
4.20 "	"	67.00 "	1 000 "

"Saturday, January 7th, 1899.—5 100 lbs. clay driven; no change in measurements taken in 8-in. pipe:

9.10 A. M.	Distance	67.00 ft.	1 000 lbs. clay.
11.25 "	"	67.00 "	1 000 "
1.25 P. M.	"	67.00 "	1 000 "
2.40 "	"	67.00 "	1 000 "
4.10 "	"	67.00 "	1 000 "

"Flow of water from 8-in. pipe clear and constant; shows no effect of charge; driving with weight of hammer only.

"Monday, January 9th, 1899.—6 200 lbs. put in; no change in measurements in 8-in. pipe taken every 1 000 lbs. All show clay at distance from top of pipe of 67 ft., or at elevation—32.44. Flow of water shows decided increase over January 7th, and runs steadily and clear, showing no effect of ramming. Weight of hammer only, required.

"Tuesday, January 10th, 1899.—4 150 lbs. driven. No change in measurement in 8-in. pipe. All show clay at 67 ft. down from top of pipe. Weight of hammer only used, no impact. Water flowing from 8-in. pipe shows slight increase over yesterday; runs clear.

"Friday, January 13th, 1899.—Clay ramming resumed to-day. Total driven, 3 750 lbs. Measurements taken in 8-in. pipe as follows:

10.20 A. M.	Distance,	67.00 ft.	1 000 lbs. clay.
12.00 M.	"	66.90 "	800 " "

"The first 1 000 lbs. drove easily; requiring weight of hammer

only, but driving seemed to stiffen until, at end of next 800 lbs., a slight drop of hammer, about 5 ins., was required.

12.40 P. M. Distance, 66.85 ft.—200 lbs. clay; still slight drop of hammer.

2.30 " " .... —250 lbs. clay. Water in pipe rose suddenly, discharging over edge in quite large volume, indicating clay has been forced upward suddenly; and after this, driving becomes easier, weight of hammer only required at:

2.45 " " 60.90 ft.—Rise 5.95 ft., 250 lbs. clay. Water was discharged as each cylinder full of clay was forced in, and continued to flow between strokes also, at the rate of about 13¼ galls. per minute until 3.15 P. M., when flow stopped, except as charge was driven.

3.50 " " 46.95 ft.—Rise, 13.95 ft. 500 lbs. clay.

4.40 " " 37.90 ft.— " 9.05 " 500 " "

"Saturday, January 14th, 1899.—4 250 lbs. clay driven. Measurements taken as follows:

9.50 A. M. Distance, 35.01 ft.—Rise, 2.80 ft. 500 lbs. clay. Water in pipe rises about 2 ins., when charge is driven, dropping back again to old level, but does not flow out of the pipe. Some water noticed coming up through back-filling around pipe, evidently from leaky joint.

10.45 " " 33.70 ft.—Rise, 1.40 ft. 500 lbs. clay.

11.20 " " 32.90 "— " 0.80 " 500 " "

11.55 " " 32.05 "— " 0.40 " 500 " "

1.10 P. M. " 32.20 "— " 0.30 " 500 " "

2.15 " " 31.75 "— " 0.25 " 500 " "

Driving now began to stiffen up and at 2.45 a slight drop of hammer, 6 ins., was required, continuing for balance of day.

3.10 " " 30.50 ft.—Rise, 1.25 ft. 500 lbs. clay.

3.55 " " 28.95 "— " 1.55 " 500 " "

4.35 " " 28.65 "— " 0.30 " 250 " "

"At this time water in pipe ceased to show any effect of driving charge. This P. M. the clay exhibited remarkable elasticity, sometimes forcing the piston back 3 ft. after driving charge.

"Monday, January 16th, 1899.—Total clay driven, 3 400 lbs. Required impact of hammer to force clay, limiting the height of stroke to 6 ins. About 45 strokes required for the cylinder, which is 6 ft. long. The clay is stiffening gradually and losing some of its elasticity, not springing back as much after each stroke. Measurements in 8-in. pipe as follows:

7.50 A. M.	Distance,	28.55 ft.	250 lbs.	clay
8.50 "	"	28.55 "	5C0 "	"
10.00 "	"	28.55 "	500 "	"
12.30 P. M.	"	28.55 "	500 "	"
1.35 "	"	28.55 "	500 "	"
3.50 "	"	28.52 "	500 "	"
4.15 "	"	28.65 "	500 "	"

"Tuesday, January 17th, 1899.—Continued driving in 3-in. pipe until noon; 1 600 lbs. driven. No rise in 8-in. pipe to-day. Clay gradually stiffening in 3-in. pipe until it requires about 90 strokes, none over 6-in. drop, to force down a cylinder full. Shifted over and started driving in 2-in. pipe at Station 5 — 97.5, 3 R., at 3.35 P. M. Weight of hammer only used, carrying clay down very slowly; 200 lbs. of clay put in.

"Wednesday, January 18th, 1899.—Continued at 2-in. pipe, Station 5 + 87.5, 3 R. 200 lbs. put in, with weight of hammer only. At noon this ceased to have effect, and a few light blows, none greater than 6 ins., were tried. Shifted pile-driver back again to 3-in. pipe in P. M. By means of water-jet, clay was removed from 8-in. pipe to a depth of about 40 ft. in order, if possible, to start clay rising in pipe again when ramming should be resumed.

"Thursday, January 19th, 1899.—Jetted out 8-in. pipe to a depth of 40.8 ft. from top when jet stopped short and seemed to bring up on small spawls or gravel. Resumed ramming at 11.00 A. M. in 3-in. pipe, using about 5-in. drop of hammer. Clay working very stiff, requiring about 240 strokes to drive charge (6-ft. cylinder); 500 lbs. put in. Total rise in pipe, 0.5 ft., from 40.8 to 40.3. Ordered clay driving stopped.

"Friday, January 20th, 1899.—Drove 100 lbs. of clay in 3-in. pipe to-day, somewhat easier than yesterday. This was done for the information of members of the American Society of Civil Engineers who visited the work to-day.

"Saturday, January 21st, 1899.—Resumed jetting in 8-in. pipe in endeavor to increase depth of jetting to at least 20 ft.

"Monday, January 23d, 1899.—Succeeded in jetting out 8-in. pipe to a depth of 60 ft. below top. At this point jet pipe brought up on

what seemed to be a bed of gravel. The flow from the 8-in. pipe increased when this point was reached.

"Tuesday, January 24th, 1899.—Resumed driving in 3-in. pipe; 350 lbs. put in by means of short drops of hammer, none over 6 ins.; 320 short drops required to force down piston (6-ft. cylinder). No rise of clay in 8-in. pipe or any indication of so doing. Ordered clay driveing stopped."

It seemed to be apparent that the cavity at the foot of the 3-in. pipe was well filled with clay, and it was probably due to some obstruction in the 8-in. pipe, such as spawls or gravel which had been forced in with the clay, that there was no longer any rise in the vertical section. The total amount of clay used in this driving is as follows:

2-in.	pipe,	Station	6 + 02,	32.4 L.	395 lbs.
2-in.	"	"	6 + 03,	17.7 L.	369 "
2-in.	"	"	6 + 02,	9.7 L.	1 012 "
3-in.	"	"	5 + 95,	12.5 R.	64 775 "
2-in.	"	"	5 + 97.5,	3 R.	400 "
				Total	66 951 lbs.

66 951 ÷ 113 = 592.5 cu. ft. or 21.9 cu. yds.

The total amount of clay thus driven into the pipes and cavity was nearly 22 cu. yds. The 60 ft. in depth of the outer 8-in. pipe which had been jettied out, was afterward filled with clay and gravel rammed in by hand, and the flow through it was stopped.

The work of tearing out the ruptured masonry due to driving clay through one of the 2-in. pipes was immediately started, and in all about 130 cu. yds. were taken out in following the fissures and cracks until they wholly pinched out. It was assumed in the beginning that the trouble lay in the joint between the upper or surface course of masonry at this point, which had been laid in Portland cement, and the masonry below, which had been laid in slower setting natural cement, and this proved to be the case; as it was found on investigating around the pipe from which the cracks radiated that its upper section and length, of something more than 5 ft. in all, had not been joined or coupled with the section below when placed, and that there was a space of 1 in. or more between the ends of the two pipes, which had allowed the clay, under the influence of the very hard driving, to force its way into the partly set mortar which surrounded the joint.

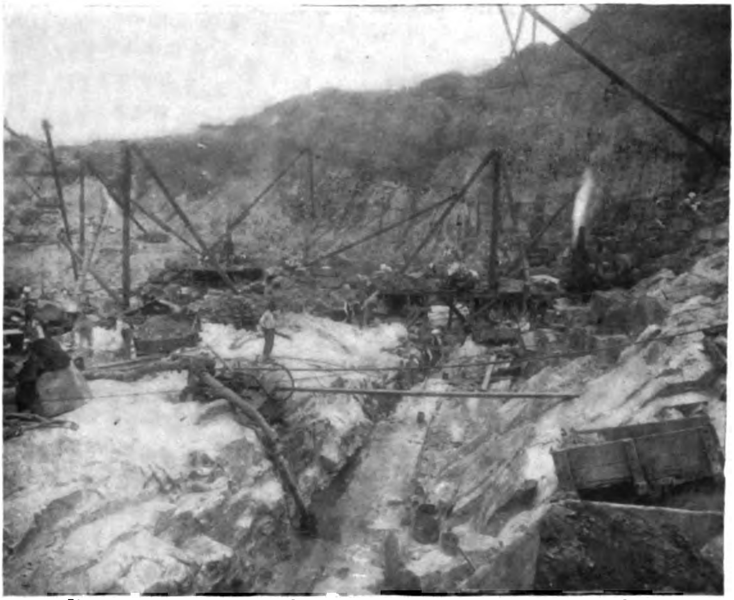


FIG. 1.—AUGUST 10TH, 1897. SEAM AT STATION 4 + 55, LOOKING EAST.

FIG. 2.—OCTOBER 22D, 1897. GENERAL VIEW FROM BERM, AT ELEVATION 115.

The course of Portland cement mortar was about 3 ft. thick. It was found that this upper joint of pipe had lifted probably 1¾ ins. from the joint below at the point of coupling. This must have taken place at first when the heavy ramming caused the rupture in that part of the pipe to which the clay cylinder was coupled, and which projected above the top of the masonry. The length of this projecting part was about 1 ft. and the parting of the joints was, of course, at the lower end and about 4 ft. 4 ins. below the masonry surface.

Adjacent to and on the level of this open joint in the pipe was the bed joint of a stone laid in Rosendale cement mortar. The stone was about 5 ft. long, 20 ins. wide and 15 ins. in thickness. The clay was found to have been forced between the under surface of the stone and its mortar bed. The clay bed was about 1½ ins. thick, and, continuing beyond the base of this stone, it rose through the joints along the sides, finding its way then along the top of the course of which the stone just mentioned formed a part, and lifting the course above, which had been laid in Portland cement mortar, and was fairly well set. The first stone mentioned was the only one laid in Rosendale cement that was found to have been disturbed in its bed, and the main crack was everywhere along the junction of the two cement mortars.

The longest horizontal radius through which the clay was found to have worked was about 5½ ft. around the pipe, and it worked up vertically through the mortar joints, and especially along the pipe, about 3 ft. The vertical cracks showing in the masonry surface were traced in some directions for 12 ft., where the width showed about 0.005 ft.; but the upper masonry course was taken out to a considerably greater distance toward and to the up-stream face of the dam, where the seepage of water through the exposed bed joints of one or two stones in this upper course indicated that the horizontal crack had extended with no vertical surface crack above to call attention to it. The clay bed, thus forced under the masonry, was found to be fan-shaped, extending 5½ ft. from the pipe in one direction and about 4 ft. sideways on each side. In the other direction its course was arrested by a large stone which offered no mortar joint that could be followed. The bed varied from 1½ ins. to ½ in. in thickness. It was found to consist of an aggregate of very thin laminations which showed clearly throughout the extent of the bed and indicated the extremely gradual way in which the rupture was produced.

Fig. 11 shows in plan and sections the area of the masonry which had to be taken up, as well as the location and extent of the clay bed, the cracks and the particular joints and stones, sketched during the work of rectifying the damage done.

The Main Dam Foundation Masonry.

The laying of the foundation masonry began on May 28th, 1896, in the bottom at Station 8 + 50, as soon as a sufficient area of bottom was ready to warrant it, and by the end of that season nine gangs of masons were at work. This involved the use of eleven or twelve derricks to allow for time lost in shifting derricks as well as for changing gangs from one point to another, owing to frequent changes in the location of sumps and subsidiary pumps which the maintenance of drainage made necessary. In the following season (1897), the number of gangs was increased to 17 on the foundation, as the season progressed, and the total number of derricks in use, including those on the side slopes for passing material from the tracks, was about thirty.

The type of derrick in general use is the "stiff leg" derrick. Having no guys, these derricks did not interfere with the cable service and they were easily moved by means of the cable, without being separated from stiff legs and platforms.

The setting of the first stone in the main dam foundation is shown in Fig. 2, Plate III. The bottom courses were laid in Portland cement (2 to 1), the vertical thickness of this work, varying from 4 ft. up, depending upon circumstances and particularly upon the amount of seepage through the fissures in the rock and the work necessary to temporarily dam up and divert such flows until the masonry was old enough and high enough to enable them to be blocked off permanently.

The rock bottom was, in all cases, very thoroughly washed and cleaned with brushes and brooms, and was then "painted" with a grout of neat Portland cement, applied with brushes, and which was allowed to set before work was done upon it. This grout was for the purpose of filling all small, fine, open cracks, seams and erosions, which were not of sufficient size or importance to warrant special treatment with the grout pump or box, and about 356 bbls. of Portland cement were used in this way on the main dam foundation and 14 bbls., up to date, on the core-wall and overflow foundations.

Owing to the extreme unevenness of the rock bottom as prepared,

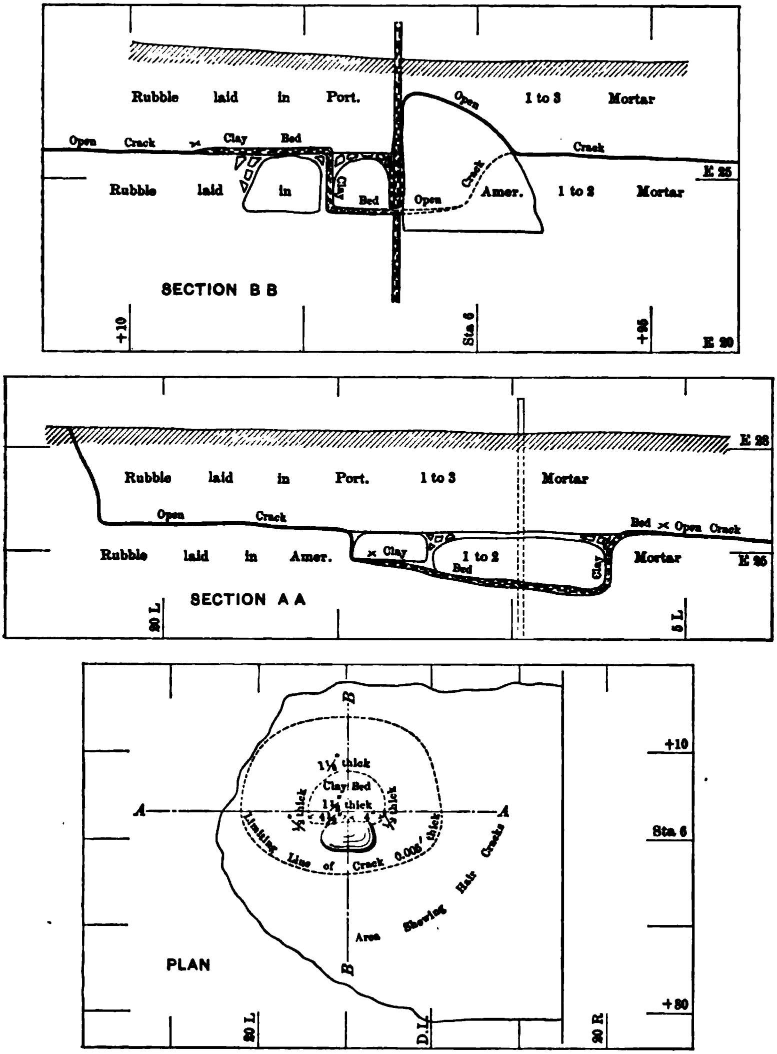

Rubble laid in Port. 1 to 3 Mortar
Open
Open Crack
Clay Bed
Crack
E 25
Rubble laid in Amer. 1 to 2 Mortar
Clay
Bed
Open
Crack
SECTION B B
+10
Sta 6
+26
E 20
E 28
Rubble laid in Port. 1 to 3 Mortar
Open Crack
Bed × Open Crack
Rubble laid in Amer.
× Clay
1 to 2
Bed
Clay
Mortar
E 25
SECTION A A
20 L
5 L
B
1½ʺ thick
Clay Bed
1½ʺ thick
½ʺ thick
A
A
Limiting Line of Crack 0.005ʹ thick
Area Showing Hair Cracks
B
PLAN
+10
Sta 6
+30
20 L
D.L.
20 R

Fig. 11.

the numerous spring holes which had to be temporarily dammed in order to divert their flows until such time as they could be choked off, and the number of grout pipes to be placed, and in some cases kept in place for months before the grouting could be done, the first season's work on the bottom masonry was done under difficulties, and the general progress up to January 1st, 1897, was between Station 7 + 12 and Station 9 + 62, covering the full width of the dam and varying in depth or height above the bottom from 10 to 40 ft. In all, about 37 000 cu. yds. were laid.

In the following year, the masonry was extended over the whole bottom, with the exception of a narrow strip near the river wall at Stations 10 + 00, used as the foundation of a trestle work in connection with the supply tracks, and a comparatively small area in the center of the wall, at about Station 7 + 50, which, during the latter part of the season, was used as a sump-hole for the main pumps, the surrounding masonry forming the sides. During that year the amount of masonry laid in the foundation was about 115 000 cu. yds., and at certain points it had risen to a considerable height, particularly over the points at which the work was started during the preceding season. The width of the foundation to be laid was about 200 ft., and the derricks were arranged in batteries of four abreast across the line of the dam, the plan being to build in racks to a convenient height and then to move the derricks forward in batteries. In this way successive racks or steps were gradually formed for the full width of the work, varying from 17 to 15 ft. in height and from 35 to 40 ft. in width or depth, horizontally, with the derricks moving from the ends toward the middle of the foundation. This arrangement is shown in Fig. 2, Plate X, but, owing to circumstances, it was not until the end of the second season's work that it could be said that the plan had been fully developed and put in complete working order.

The faces or step courses of these racks were limited to rises of 3 ft., with about the same treads, making the slope of the rock nearly 1 : 1. Care was taken to avoid long straight joints across the dam, between successive racks, by varying the lines of their faces at intervals with "scallops" or heavy "returns."

By the end of the second season, the main foundation had risen high enough above the bottom to be drawn in to its neat lines at all points and to afford a parapet which retained the wash from

FIG. 1.—MARCH 3D, 1899. DAM, FROM HILLSIDE NEAR SPUR ROAD, LOOKING SOUTH.

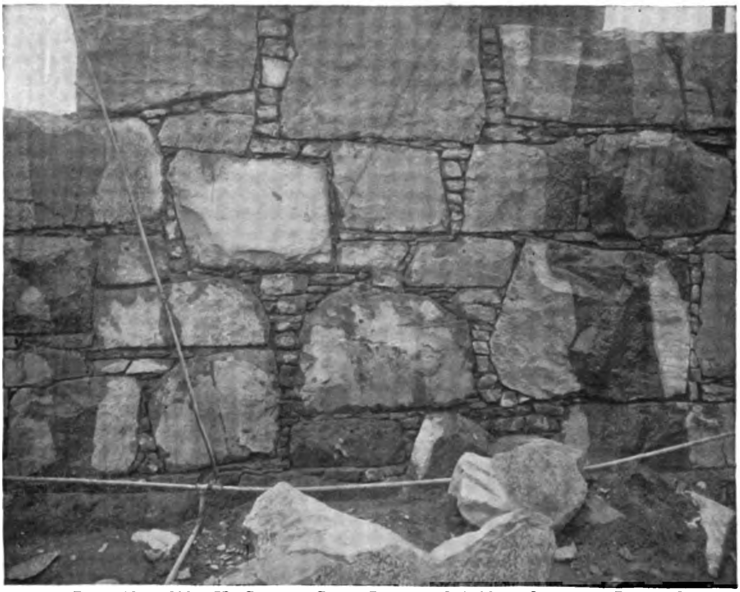


FIG. 2.—JUNE 22D, 1898. UP-STREAM SIDE, STATION 8 + 70 ±, SHOWING JOINTS RAKED OUT FOR POINTING.

the earth slopes and enabled the refilling work to be started. On the up-stream side, the masonry face was drawn to its neat lines as soon as practicable upon leaving the rock bottom, though this involved a considerable depth of re-fill between the masonry and rock face, particularly at the deeper points where the rock was disintegrated and had broken back of the excavation lines.

On the down-stream side the masonry toe was built solid to the rock face up to the surface, when it was drawn in to the neat lines planned. This junction with the rock face was made still more compact, as noted in the description of the "grouting," by filling with grout the eroded seams showing in the rock face as the toe masonry was built up.

As above stated, the courses adjacent to the rock bottom were laid in Portland cement, a 2 to 1 mortar mixture being used. The special purpose was to obtain a quick-setting mortar and thus avoid, to as great an extent as possible, any wash or trouble from seepage and flows through the bottom which had been choked off. Above these courses, and for the great bulk of the warm season's work, American cement, mixed 2 to 1, was used. During the winter months, Portland cement, 3 to 1 mixture, was substituted for the American cement, and work was carried on steadily on pleasant days when it was not too cold.

Care was taken to lay no masonry on days when the temperature was steadily below the freezing point, and on cold nights and mornings brine and warm water were used in mixing mortar, and the sand during the whole season was heated and dried in large boxes furnished with steam coils arranged for that purpose. Care was also taken to cover fresh work at night with brine, salt and canvass, and to thoroughly clean its surface and joints in the morning with steam and hot water in order that all frozen dirt and mortar scale might be removed. All stones and spawls used in cold weather were also thoroughly cleaned and washed, and thawed out with hot water and steam; pipes for both being provided for each gang of masons employed.

The stone used for the rubble masonry is quarried from a rocky hillside in the Valley of Hunter's Brook, a tributary of the Croton River, at a point about 2 miles above the dam. This stone is classed by geologists as "gabro" rock, or, commercially, as a dark colored granite, although it is without quartz and has a large amount of hornblende in its composition. It is very hard and tough, as well as heavy, and

weighs 185 lbs. per cu. ft. The quarry is connected with the dam by a railroad, and the stone is quarried and sent down in large blocks varying in size, ordinarily, from 1 to 3 cu. yds., although the greater limit is not reached commonly. Stones even of larger size have been furnished occasionally, but difficulty in handling them renders such sizes undesirable.

The spawls and small "chunks" are furnished from quarries along the line of the railroad nearer the dam, and are of the country rock, a laminated gneiss.

In laying the stone, care is taken to see that each stone has been thoroughly cleaned and washed with water in summer and steam in winter. The stone is bedded in a heavy bed of mortar in which flat spawls have been placed to "make up" to such hollows or deficiencies as may be apparent in the bed of the stone. The stone in question is then raised, and the imprint it has made in the mortar bed is used as a guide to complete the necessary making up; additional mortar is then placed over the new bed and the stone is lowered again into place, care being taken to place it exactly as it was before. It is then shaken down by bars placed successively at different ends of the stone until the mortar underneath is pressed out on all sides, when, if it is apparent that it "floats" freely without touching the spawls or stones below, it is allowed to remain. Should there be any doubt about this, however, it is taken up a second time, or as often as is necessary to insure a thorough and tight bedding, well made up and with the minimum of mortar left in necessary to the result wished.

The spaces around these stones are then carefully filled with mortar into which smaller stones and spawls are hammered, care being always taken that no small stone shall be hammered into place unless there is an ample bed of mortar under it. All old work is thoroughly cleaned with brooms and washed with water before fresh work is built upon it. It is also carefully sounded with iron rods to make sure that no small stones or spawls have been loosened in the bed. In cold weather the precautions necessary in building on old work are greater, as mortar more or less frozen and disintegrated is commonly found on the surface and the dépth of spawls liable to be loosened is much greater.

The general character and appearance of the masonry in the racks is shown in Fig. 2, Plate XII. The stones were of such size that an average rise of 3 ft. in the courses was readily maintained. On the

FIG. 1.—MARCH 27TH, 1899. SPILLWAY AND CROTON RIVER. LOOKING EAST.

FIG. 2.—MAY 27TH, 1897. MAIN DAM MASONRY.

down-stream side the batter called for by the theoretical sections was obtained by stepping. For this purpose selected stones were of course necessary. The quality and appearance of this work are shown in Fig. 1, Plate XI. The steps were laid out with rises of from 24 to 30 ins., the latter limit preponderating, and the whole of the step is built outside the neat batter line.

On the up-stream face all joints were raked out 2 ins. in depth and were then pointed up with Portland cement, mixed 1 to 1. This includes also the core-wall and spillway masonry as well as the foundations of the main dam. Fig. 2, Plate XI, shows a section of the up-stream face of the main dam, in which the joints have been raked out and are ready for pointing.

The foundation masonry laid to date in the spillway was laid in 1895, since which time nothing further has been done. It is shown in Fig. 1, Plate XII. The masonry work of the lower part of the core-wall was begun early in the history of the dam work at the extreme south end, and was so prosecuted that by the time the masonry of the main dam foundation had reached the point of junction with the core-wall there were only 100 ft. of wall foundation to be done to complete the connection. This has since been done, and the wall is being built up to the surface of the trench excavated for it.

The refilling against the main dam foundations calls for no special comment except in one instance where, on the up-stream side between Station 6 + 12.5 and Station 6 + 62.5, the bad rock forming the face of the excavation at this point continued to fall into the pit, breaking back of the original excavation lines, after the excavation had reached the bottom, and while the up-stream face of the masonry was building. By the time this masonry had reached a height sufficient to be out of danger from the gradually falling rock, a large mass of the latter had fallen in behind the wall at the bottom into the space which during this time had been used as a sump. It was impracticable to get this material out, as the overhanging rock made it too dangerous for men to attempt it without very thorough protection from above, and it was at the same time extremely desirable to have at this point and elevation especially compact back-filling, rather than a large quantity of loose rock.

The plan and sections in Fig. 10 show the extent to which the rock broke back of the lines and the amount and extent of the broken rock

or debris which fell in from above while the masonry wall was being carried up.

This space was used as a sump-hole, the suction pipe being kept at a low elevation at about Station 6 + 20, at which point the rock slope stood. The water flow through the face of the rock was very free, as the face was full of open fissures, particularly through that part of the face shown in the plan as furthest to the right. This formed quite a recess, and through it came at this time most of the flow pumped from the up-stream side of the dam.

This water flow brought a large amount of fine silt with it, and it was reasonable to suppose that the debris at the bottom and along the wall was, in the course of time, filled with it. It was found impossible to force pipes through the debris to the bottom to test this, and two holes were therefore drilled through the masonry at such positions and angles as to reach low points in the filling. These holes are shown on the plan and sections. Drilling them was a matter of considerable difficulty, but they were finally forced through, but filled immediately with fine silt forced in from below, and no grout could be pumped into them. Two other holes were started in this vicinity, for the same purpose, but could not be carried through.

In making further attempts to prove the compactness or otherwise of this mass of debris the surface shown in the plan to the left of the dotted line was covered with several feet of fine sand and gravel after the pipes indicated had been forced into the mass as far as possible, which in some cases was 5 or 6 ft.

The inflowing water, as stated above, showed itself particularly in the space to the right of the dotted line, in which no gravel was placed. The pump suction back of the wall at about Station 6 + 00 kept the water well below the general surface of the debris which had been covered with gravel, and into three of the eleven pipes 17½ bags of American cement (1 to 1 mixture) were pumped as shown. The other pipes, which were tried later, would take nothing, and the job was finally completed by pouring a large amount of grout, 30 bags of American cement (1 to 1 mixture), into the water space back of the dotted line. This pouring was kept up until the water flow was stopped at the point under treatment, and was forced through other seams in the rock face more directly to the sump-hole which was back-filled later with gravel and sand in the ordinary way.

The Pumping for the Main Dam Foundation.

The work of pumping began in April, 1895, a 10-in. two-cylinder Worthington pump being installed at first. It was placed as near the sump as possible, and was fed from boilers placed at the top of the slope on the down-stream side, not far away. Two 100 H.-P. boilers were installed at this time. This number was increased later to four in all, and at certain times, when the demand for steam for the pumps was heavy, but little outside use was made of the boilers, although the excess in boiler power was at least one, when they were all working at their full capacity. As a rule, however, the whole four were kept continuously in use, working moderately after the 10-in. pump had been replaced, a few months after its installment, by a 12-in. compound, double-cylinder pump of the same make, to which two others of the same size were added later.

With this force of three large pumps two were kept at work at moderate speed, while the third was held in reserve, and the 10-in. pump kept either as additional to the reserve or at times used in connection with a number of smaller auxiliary pumps which were constantly in use during the excavation work and until the foundation masonry was complete, pumping from various points in the bottom to the main sump. From the beginning of pumping operations until November, 1898, the main pumps were kept on or near the lower or down-stream slope of the main cut, and the sump was maintained near by, either on the natural bottom, or, as happened during one winter, in a large hole left in the bottom masonry at a low elevation near the down-stream toe of the dam.

It was extremely inconvenient at times to have to limit main pumping operations to one point and to be obliged to lift all the water from the auxiliary pumps, in some cases over the low-lying portions of recently laid foundation masonry, but the risks to main steam pipes laid across the dam would have been too great, either before or after the beginning of the mason work, while the water flow was large. The discharge was ordinarily through a system of four pipes 12 ins. in diameter, two of which were laid through the lower wing-dam at a low elevation and two through the river wall at a somewhat higher elevation. By this means a considerable lift was avoided, the top of the wing-dam being in the first case about 20 ft. above the pipe openings.

TABLE No. 1.—MEAN MONTHLY TEMPERATURES OBSERVED, IN DEGREES, FAHRENHEIT.

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	APPROXIMATE LOCATIONS OF OBSERVATIONS.					
Date.	8 + 50 / 195 L. 8 + 10 / 195 L.	6 + 50 / 36 R. 6 + 00 / 30 R. 5 + 30 / 30 R.	7 + 60 / 200 L.	7 + 70 / 30 L. .... Cave.	7 + 00 / 12 R. 6 + 00 / 70 R.	In Channel. River.
	°	°	°	°	°	°
Feb., 1896.......	44.1	50.2	....	....	....	35.8
Mar., "	42.3	50.8	....	....	....	36.0
Apl., "	45.0	51.0	....	....	....	51.5
May, "	50.0	52.7	....	....	....	....
June, "	51.5	54.0	....	....	....	....
July, "	56.7	53.3	51.3	....	....	82.0
Aug., "	61.0	52.0	60.0	....	....	72.0
Sept., "	63.0	51.5	65.0	62.0	....	73.0
Oct., "	67.0	58.0	66.0	64.0	....	....
Nov., "	64.7	....	60.0	60.0	....	56.0
Dec., "	56.6	52.5	62.8	55.6	57.6	35.6
Jan., 1897.......	55.0	51.0	57.0	53.0	55.0	34.0
Feb., "	50.0	50.0	50.0	50.0	53.0	....
Mar., "	46.0	48.7	48.7	48.7	52.0	40.3
Apl., "	42.7	48.0	45.3	47.3	51.3	....
May, "	48.0	....	48.0	48.0	51.0	65.0
June, "	64.0	56.0	58.0	54.0	54.0	76.0
July, "	65.0	56.5	64 0	58.5	56.2	76.0
Aug., "	68.7	60.7	64.7	62.0	61.7	76.0
Sept., "	71.2	62.7	71.2	66.2	66.2	74.2
Oct., "	69.6	64.4	69.6	64.8	65.2	64.5
Nov., "	66.5	63.5	67.0	63.0	64.0	47.7
Dec., "	61.6	61.0	63.6	59.2	62.0	40.0
Jan., 1898.......	56.0	57.0	59.5	57.5	59.5	38.7
Feb., "	46.0	52.0	53.0	52.2	54.7	37.2
Mar., "	43.0	52.2	49.0	51.0	54.4	47.6
Apl., "	45.0	50.7	46.2	49.5	51.7	51.0
May, "	49.2	52.0	49.5	50.0	51.2	60.5
June, "	54.4	54.4	55.4	51.6	50.2	74.4
July, "	61.7	59.0	58.7	53.2	51.0	80.0
Aug., "	64.1	60.0	62.0	56.2	53.5	78.2
Sept., "	68.0	62.3	65.0	59.8	54.4	76.2
Oct., "	68.2	57.0	62.5	60.5	54.0	63.5
Nov., "	68.0	55.3	57.0	....	58.0	49.7
Dec., "	66.0	....	66.0	....	58.0	39.0

Valves at the outer ends of the lower pipes secured the work from the possible danger of back flow when the water was high in the river. In December, 1898, one of the main pumps was placed on the up-stream side of the dam wall, which by this time had been carried all the way across the valley and up to a considerable height above the bottom, and two were kept on the down-stream side, steam for the former being carried across the foundation wall. By this time the amount of water to be pumped had decreased materially, as a large

amount of back-filling had been done on both sides of the dam, and the elevations of the sump-holes had been raised.

During the progress of the main dam excavation it was early noticeable that the flow into the sump-holes seemed to come from particular points on the gravel slopes, following the toes of the slopes down, as the depth of excavation increased. This was noticeable on both the up-stream and down-stream sides, and the flows or springs continued to be identified easily as the work progressed and the outline of the foundation masonry was completed, and the flows or springs on the upper or lower side were separated. These flows were confined to the gravel slopes, through which they came freely, and as the back-filling was gradually raised they were forced back up the slopes to the vicinity of the points where they had originally shown themselves.

At the south end of the main cut and along the sides near the end, where the slopes were nearly all hardpan, practically no water was encountered, although there was a considerable area of gravel and boulder slope under the hardpan near the south end of the side slopes on the "quarters." There was, however, a considerable seepage through the lower half of the very high hardpan slope at the south end which, particularly in winter, through the frost and thaws, caused a good deal of gradual sloughing off of the bank, although at no time was the amount of seepage enough to cause a definite flow from the slope.

A long series of observations of the temperatures, Table No. 1, taken at the points where the flows were best defined, is of interest as indicating, perhaps, some differences in the causes and origins of the various flows observed.

In Table No. 1 are shown six series of observations, including one in the river channel. In Column No. 2 the observations were of a flow on the down-stream side of the main cut, at the point nearest to the up-stream toe of the lower wing-dam. In Column No. 3 the flow observed was near the point on the up-stream side where the heavy flow or spring at Station 5 + 95 was in time developed, and it is assumed that this flow was the same, practically, that showed in the spring when it was reached. In Column No. 4 the observations are of a flow developed on the down-stream slope some distance from the flow in Column No. 2. In Column No. 5 the flow was from the large cave at Station 7 + 70 ± on the up-stream side. The flow of Column

No. 6 was also on the up-stream side, and the observations were taken at first in a large well-hole which was built, temporarily, near the up-stream face of the masonry, and later from the flow which showed outside the masonry line after the well was filled up and the water forced outside of the masonry limits.

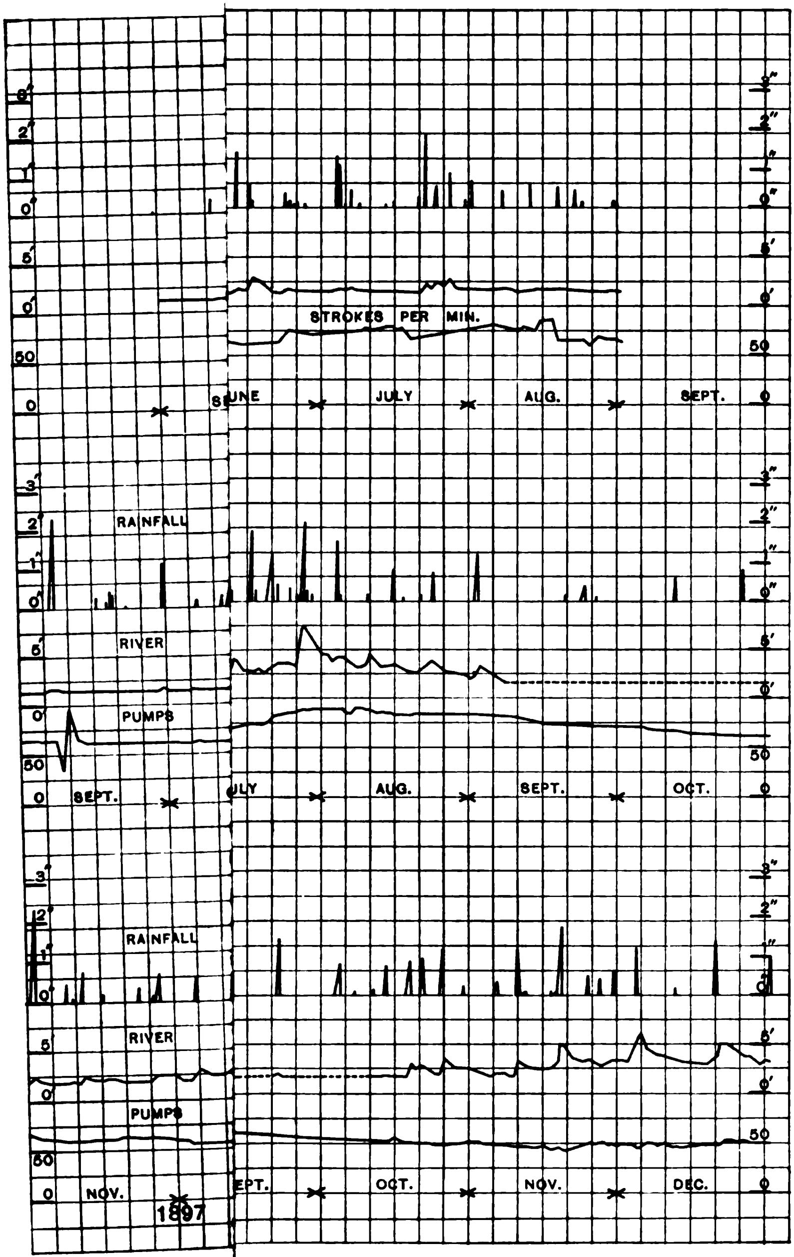
The stations at the heads of the various columns show the approximate locations of the points of flow at which the temperatures were taken. In some cases there were variations of location at intervals owing to the shifting of the springs from various causes, such as a deepening of the excavations in the vicinity, or, as in the case of the well mentioned in Column No. 6, some change in the masonry and the channels left temporarily in it. Column No. 7 shows the temperature of the water in the river.

An examination of these observations shows clearly a certain uniformity in the flows from the excavation, particularly in regard to the times of extreme temperatures, which occur in September or October and March or April. It is evident that there was no direct connection with the water flowing in the river, and that the two springs observed on the down-stream side correspond closely, as might have been expected, while the other three springs located on the up-stream side are uniform in showing less extremes in temperature and also a close correspondence with each other. In the river the extreme temperatures shown were in January or February and July.

The flow observed in Column No. 3 shows, however, but little variation during the first twelve months, quite in contrast to the others. The observations during these months were evidently of the water from the heavy spring which, when solid rock bottom was reached, was found to flow from the large erosion at Station 5 + 95, 12 R. The point at which the temperatures were taken in this case was some distance from the spring hole as finally defined, and the water was then piped for some months directly to a subsidiary sump.

Temperatures of the water, however, continued to be taken as it flowed from this pipe and later from the pipe used to divert the spring flow to the up-stream side of the masonry. The elevation of the outlet of this pipe was increased gradually as the masonry and the backfilling rose, and the observations were continued until the clay driving had stopped the flow of the spring. The observations from January, 1897, showed variations which correspond with the variations observed

PLATE XIII.
PAPERS AM. SOC. C. E.
JANUARY, 1900.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



at the other points of flow, although they were not so marked in degree. As this spring was quite different in its characteristics from any of the others coming originally from a much greater depth and with the location of its flow confined to one place during all the time, it is an interesting question as to why, after a year's nearly steady temperature, variations corresponding with the surrounding springs should develop. It may be that after the first twelve months this spring flow was, to a certain extent, exhausted, and its temperature was affected in a greater degree by the increasing proportion of ground-water near its outlet. Its evident decrease in flow, as time passed, may be one argument in support of this assumption.

A daily record of rainfall, river flow and pumping is shown on Plate XIII. The rainfall and river gauge readings are shown from September 1st, 1895. The pumping record is begun in October, 1895. This shows irregularities for the succeeding eight months, which are partly due to lack of systematic observations and records during that time, and partly to the frequent changes in the location and elevation of the pumps, as it was during these months that the increase in the depth of the sumps was the most marked. The duration of each rainfall is indicated as nearly as possible by the span of the bracket, the height showing the total fall in inches. The flow in the river is indicated by observations taken at a gauge a short distance below the dam location, and the diagram shows the depths of the flow at that point in comparison with the low-water elevation, which is about at gauge reading 1.70. The pump diagram is based on the average daily speed, in strokes per minute, of one 12-in. pump, and all observations within the above time limits have been commuted to this basis, as the only one by which a direct comparison of the pumping work from time to time could be had.

As to the actual amount of water pumped, various tests of the pump capacity were made from time to time. The 10-in. pump was stated by the makers to have a capacity of 1 500 000 galls. per 24 hours, with a maximum rate per minute of 36* strokes. Each of the 12-in. pumps, at the same maximum rate of speed, had a capacity of

* Double strokes, or one complete revolution, or "cycle," of the pump action. The capacity of each 12-in. pump cylinder for one complete stroke was about 29 galls., the diameter of the piston being 17 ins. and the length of stroke 15 ins. The 12-in. pumps were so designated to agree with the diameter of the discharge outlet.

4 000 000 galls. per 24 hours. The tests of the actual amount of water pumped are as follows:

On Sunday, October 27th, 1895, water in the main cut was allowed to rise from Elevation 7.81 to Elevation 10.14, the limits between these elevations having been carefully cross-sectioned. The amount of inflow was estimated as 814 275 galls. in 7 hours, or at the rate of 2 800 000 galls. per 24 hours.

The pumps stopped at 8.40 A. M., and resumed work at 3.40 P. M. The water during this interval rose 2 ft. 4 ins., or from elevation 7.81 to 10.14, and seemed to rise at a constant rate of nearly 4 ins. per hour.

* The small Worthington pump (1 500 000 galls.) was unable to control the water in the main cut below Elevation 13. The water gained on the pump up to this elevation, but was then held constant by the small pump.

On December 19th, 1898, an experiment on the pumping capacity of one 12-in Worthington pump was made. The result showed a capacity of 50 galls. per stroke.

The experiment was made by the use of a large sump-hole on the down-stream side of the main dam. One side of this sump was formed by the wall of the dam, the others by the back-filling in progress at the time the experiment was made. The following is a *résumé* of the results. The sump had been carefully cross-sectioned.

DECEMBER 19TH, 1898.

Calculations to determine efficiency of pumps and amount of water flowing into sump. 12-in. pump, down-stream sump.

Capacity of sump between Elevation — 2.2 and Elevation — 8.0	287 512	galls.
Experiment 3.20 P. M. to 7.10 P. M. 230 minutes.		
At 3.20 P. M. the sump was empty, pumps shut off.		
By 7.10 P. M. the sump had been filled by water flowing in from springs. In 230 minutes the amount of water flowing in equals capacity of sump	287 512	galls.
Flow per minute	1 250	"

* Note taken at time above experiment was made.

Experiment 10.30 A. M. to 2.27½ P. M.

237½ minutes.

Sump full at beginning, empty at end.

950 815 Gauge reading at 2.27½ P. M.

939 181 " " 10.30 A. M.

11 634 Number of strokes of pump during experiment.

49 Number of strokes of pump per minute.

Flow during experiment

= 1 250 galls. × 237½ = 296 875 galls.

Capacity of sump	287 512	"
Amount pumped	584 387	"
Divide by number of strokes (11 634) and we get	50.23	" per stroke.

Experiment 2.27½ P. M. to 3.20 P. M.

52½ minutes.

Sump empty at beginning and end.

952 453 Gauge reading at 3.20 P. M.

950 815 " " 2.27½ P. M.

1 638 Number of strokes of pump during experiment. Multiply by 50.23 galls. per stroke and we get

82 277 galls. pumped out during experiment equals gallons flowing in

82 277 ÷ 52.5 = 1 567 galls. Flow per minute (when sump is empty)

1 638 ÷ 52.5 = 31.2, average number of strokes per minute.

Flow per minute with sump empty during experiment	1 567	galls.
Flow per minute with sump empty at beginning and full at end, or *vice versa*	1 250	"
Difference	317	"
Capacity of pump per stroke from experiment	50.23	"
Capacity of pump per stroke (pump measurement)	58.00 ±	"

N. B.—For 24 hours previous to these experiments the water in the sump was held at Elevation — 5.54, and the speed of the pump averaged 29.5 strokes per minute. The rise and fall of the water in the sump in the above experiments was between Elevations — 2.2 and — 8.0.

At 49 strokes (double) per minute the amount pumped is at the rate of 3 550 000 galls. per 24 hours. As the maximum number of strokes shown on the pump diagram is not more than 90 strokes per minute, it may be assumed that the maximum flow into the pit was less than 7 000 000 galls. per day. 29.5 strokes per minute equals about 2 160 000 per day.

A special experiment was made at a time when one 12-in. pump was at work on each side of the dam, in order to detect if possible any variation in the pumping rate owing to the relative difference in the elevations of the sump levels. The result is as follows:

TABLE No. 2.—Calculations to Determine Effect of Elevation of Water in Sumps on Up-Stream and Down-Stream Sides of Dam upon Amount of Water Pumped.

Experiment, December 21st, 1898.

Elevation of water in down-stream sump — 6.55.
" " " up-stream sump + 8.7.
12-in. pump used in each sump.

Time.	Down-Stream Sump.			Up-Stream Sump.
	Register.	Difference.	Strokes per minute.	Strokes per minute.
7 A. M.	017 256	....	....	....
8 "	018 968	1 712	28⅔	21
9 "	020 726	1 758	29⅓	19½
10 "	022 415	1 689	28	20½
11 "	024 457	2 042	34	20
12 M.	025 883	1 426	23¾	21
1 P. M.	027 601	1 718	28⅔	21½
2 "	029 306	1 705	28⅓	21½
3 "	031 030	1 724	28⅔	21½
4 "	032 739	1 709	28½	21½
5 "	034 462	1 723	28⅔	21½
6 "	....	....	....	21½
Total strokes			286⅓	231
Average strokes per minute during day			28.6	21.0

Water in down-stream sump lowered during night.

EXPERIMENT, DECEMBER 22D, 1898.

Elevation of water in down-stream sump — 8.01.
" " " up-stream sump + 8.7.
12-in. pump used in each sump.

Time.	Down-Stream Sump.			Up-Stream Sump.
	Register.	Difference.	Strokes per minute.	Strokes per minute.
8 A. M.	061 265	....	....	19½
9 "	062 908	1 643	27⅓	19½
10 "	064 589	1 681	28	19½
11 "	066 252	1 663	27¾	19½
12 M.	067 893	1 641	27⅓	20
1 P. M.	069 535	1 642	27⅓	19
2 "	071 167	1 632	27⅓	19
3 "	072 807	1 640	27⅓	20½
4 "	074 425	1 618	27	19½
5 "	076 046	1 621	27	20
6 "	....	....	....	21
Total strokes			246⅓	217
Average strokes per minute during day			27.4	19.7

CONCLUSION.—The relative heights of water on the up-stream and down-stream sides of the dam seem to have no effect on the relative amounts of water pumped.

NOTE.—On December 20th about 0.35 in. of rain fell, which may account for the greater amount of pumping on the 21st than on the 22d.

It is not assumed that the foregoing tests are accurate. They are sufficiently reliable, however, to enable it to be said that the maximum daily pumping did not at any time exceed 7 000 000 galls., and the pump diagram furnishes a reliable comparison between the amount done from time to time and the maximum. These diagrams, it may be said, are chiefly of interest in that they serve to show the relations existing between the rainfall and the resulting river flow and necessary pumping. As to whether any deductions of value can be made, excepting that in a gravel bottom, below sea level, in close proximity to a river large in times of heavy flow, the amount of water pumped was under 7 000 000 galls. per day, while the area of the pump well was at least 3 acres, and the depth 130 ft., remains to be seen.

Plate XIV shows curves deduced by averaging per month the various data shown in Plate XIII. On it are also shown the curve of increasing depth of sumps from which the pumping was done, and a curve showing the increase in yielding volume as the depth of the sumps and the size of the excavation increased. These curves are not extended beyond March, 1898, as by that time the refilling had been well started,

while the whole foundation had been covered with masonry which had been carried up to a considerable height above the bottom.

The curves show, as might have been expected, the effect of the rainfall upon the river which for most of the time is closely corresponding, although the river may be somewhat slower in its action. This does not hold good for the dry summer and autumn months of 1895 and 1896, but the correspondence is close in 1897 when extreme low water was not reached until September. The comparatively heavy flow in February, 1898, however, does not seem to be accounted for by any special rains at about that time.

The pump curve shows a constant rise in 1895 and to May, 1896, as the depth and yielding volume increased. The apparently extra rise in this curve from January to April, 1896, may have been influenced by the rainfall, to which it seems to have responded more quickly than the river. Again, in July and August, with constant depth, there is shown a quick response to the rainfall which is not noticed in the river. In December the pumping had fallen off materially, although the depth and yielding volume were on the increase. The gradually diminishing rainfall, from September to December, may possibly account for this. The steady increase in the rain from January to July, 1897, is noticeable in the river curve and marks a constant increase in the amount of pumping until the maximum of yielding volume and depth is reached. The falling off in pumping from July to December, corresponds fairly, although, perhaps, a month behind in time, with the rain and the river curves, but results from December to March, 1898, are due partly to the influence of the back-filling, which must have begun to make itself felt, and partly, doubtless, to the fact that the yielding volume was beginning, as it did in the previous year, to show signs of being pumped out at the end of the dry season.

It seems fairly conclusive that the flow in the river had, on the whole, but little direct influence on the pumping. In other words, the wing-dams were efficient in stopping anything like a direct leakage or flow from the river to the excavation pit. Another conclusion is that a considerable time elapsed between the rainfall and its effect on the pumps, amounting in certain cases to as much as two months.

The data from which the curve of yielding volume is obtained are due to calculations which show approximately at the end of each two

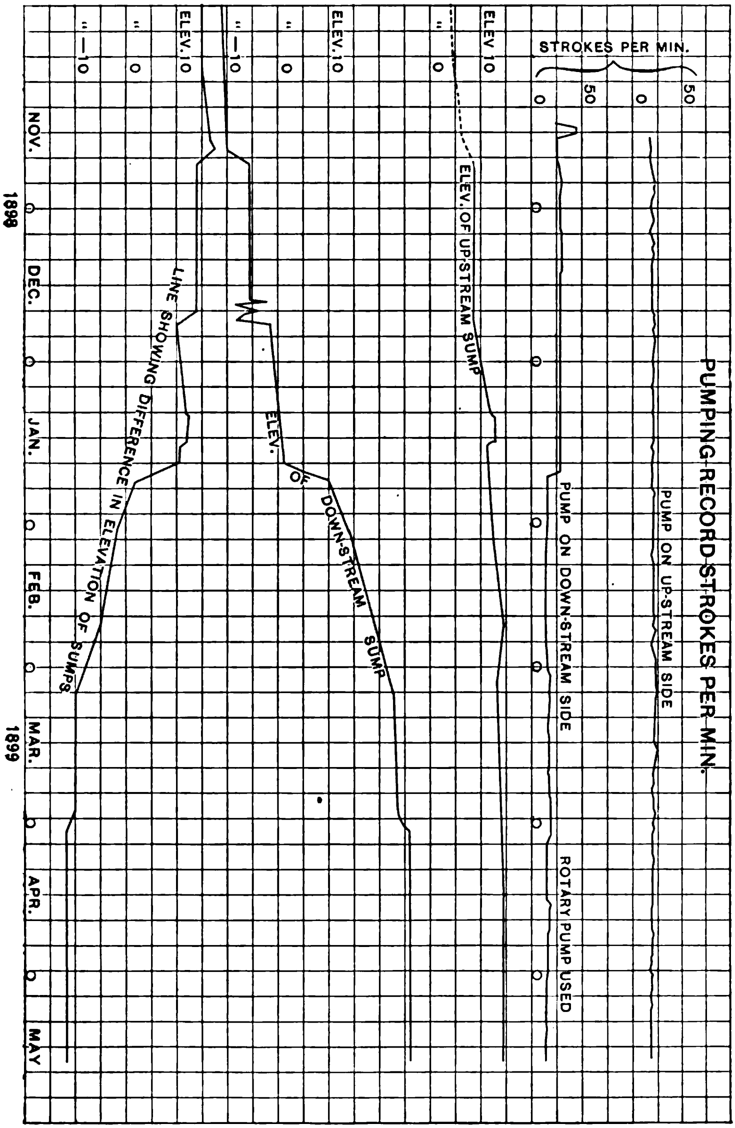

PUMPING RECORD STROKES PER MIN.
STROKES PER MIN.
0
50
0
50
PUMP ON UP-STREAM SIDE
PUMP ON DOWN-STREAM SIDE
ROTARY PUMP USED
ELEV. 10
ELEV. OF UP-STREAM SUMP
ELEV. OF DOWN-STREAM SUMP
LINE SHOWING DIFFERENCE IN ELEVATION OF SUMPS
NOV.
DEC.
JAN.
FEB.
MAR.
APR.
MAY
1898
1899

FIG. 12.

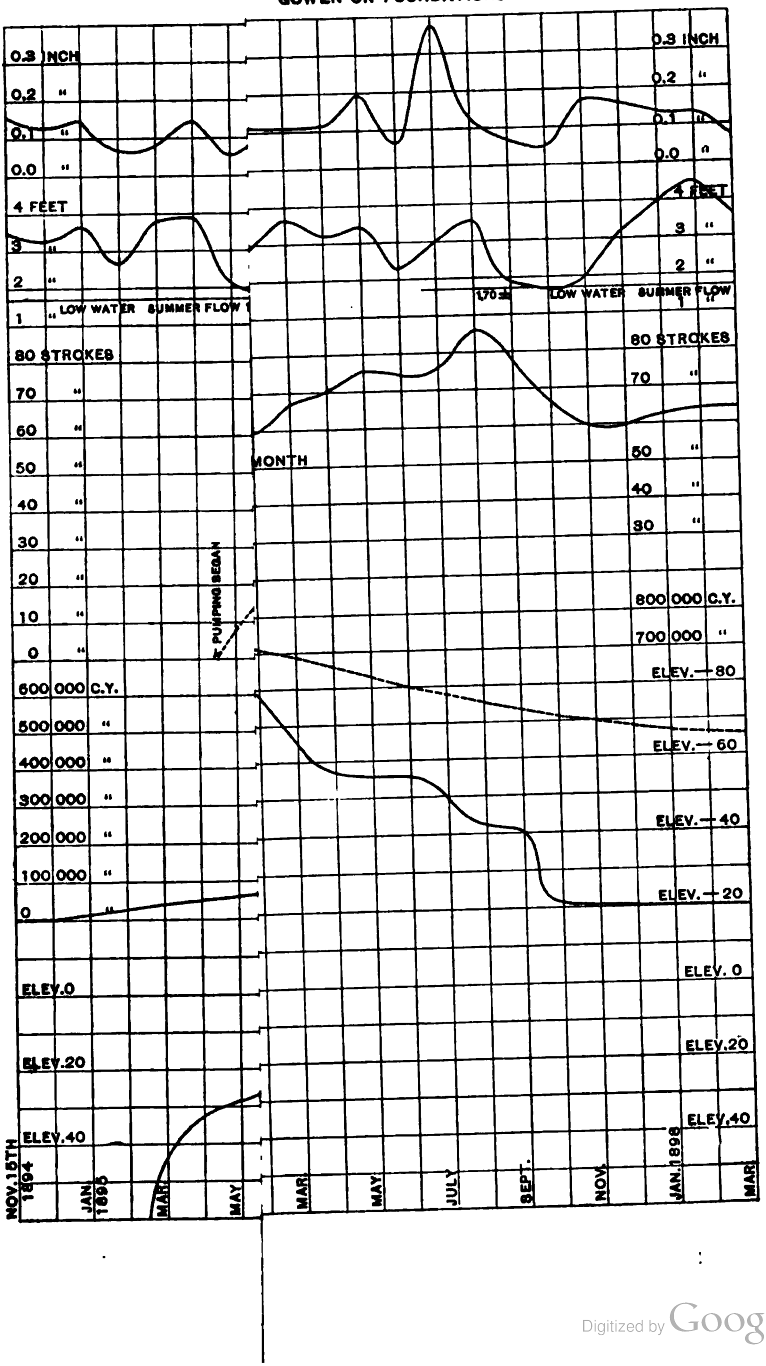
months the amount of sand and gravel on the slopes of the pit which furnishes water storage space. It was assumed that all space in the slopes above an angle of 20° with the horizontal, the vertex being taken at the lowest point in the section excavated and kept clear by the pumps, would yield water, except those parts of the slopes which were formed of hard-pan and which were not included in the calculations of volumes.

Fig. 12 shows two diagrams of pumping commuted to equivalent strokes of a 12-in. pump, covering the time from November, 1898, to May, 1899. One of these is the record of the pumping from the up-stream side of the main dam, which was done wholly by a 12-in. pump during that time.

On the down-stream side other pumps were used, as noted on the diagram, but some careful experiments were made by which the relative amount of pump work done has been fairly commuted to the 12-in. standard. The accompanying diagrams, which show in both cases the mean elevations of the two sumps from time to time and at the same time the difference in elevation of the water on the two sides of the dam, certainly do not indicate any connection between the two sump-holes. The up-stream pump shows a very slight increase in March and a very gradual decrease to May 15th, with a gradual rise in water elevation to the middle of February, and no material change later. On the other hand, the down-stream pumping shows a material diminution about February 1st, with many changes of water surface and a very material rise in the sump elevation at about the same time.

The relative elevations of these sumps varied during these months decidedly, the up-stream sump changing from about 17 ft. above the sump on the lower side, to an extreme of about 10 ft. below, most of the change, however, taking place on the down-stream side, as shown. These records are suggestive, in view of the general character of the limestone foundation and the possibility of the presence of open seams and channels below those treated in preparing the bottom for the masonry. With a head of nearly 20 ft., long sustained on the up-stream side, then varied gradually until the head on the other side was 10 ft., there seems to have been nothing, in this reversal of relative conditions, to affect the flow of water on either side.

PLATE XIV.
PAPERS AM. SOC. C. E.
JANUARY, 1900.
GOWEN ON FOUNDATIONS OF NEW CROTON DAM.



General Remarks.

While the purpose of this paper is to deal particularly with all the various features of this work which pertain directly or indirectly to the foundations of the dam, dwelling in some cases in considerable detail upon certain points, it is evident that much information, some of it of interest if not of considerable importance, relating to the general design, construction and progress of the work has been necessarily omitted. In fact, anything approaching a comprehensive account of these matters would require a book of ample size to give the subject adequate treatment. However, a few figures are added here relating to certain prominent features which have not been noted or described previously.

The main feature of the construction work is the rubble masonry. The total amount required will be not far from 650 000 cu. yds., and of this, at the date of writing, about 405 000 cu. yds. have been laid. The contract price for this item is \$4.05 per cubic yard when American cement mortar is used. This price is increased to \$4.94 and \$5.35 per cubic yard, as Portland-cement mortar (3 to 1) or (2 to 1), respectively, is used. The facing stone masonry, of which it is expected that at least 24 000 cu. yds. will be used, forms another important item. This is used for both the up-stream and down-stream faces of the main dam and overflow above the lines or elevations to which the refilling and embankment will be carried. This facing stone is cut in courses which vary in rise from 15 to 30 ins., having a uniform depth of bed and build of not less than 28 ins. Headers, of which every third stone in each course is one, are not less than 4 ft. in length, and are used to insure a bond with the rubble backing or hearting.

All joints of this stone are cut to lay to ½ in. in width from the face back for 4 ins. in depth on the sides and beds. For the remaining depth the stones must be cut full, to joints not exceeding 2 ins. in width between adjoining stones when laid. In this way there is insured a moderately fine outer joint which is thoroughly raked and pointed to a depth of 2 ins. or more, while the wider 2-in. joints give an opportunity for any settlement that may possibly occur in the future due to inequalities between the relative composition of the facing as compared with the backing stone to which it is bonded.

On this facing stone it is depended to insure the practical watertightness of such parts of the structure as are exposed directly to

water pressure. This stone is laid in Portland-cement mortar, (mixed 2 to 1), and in the pointing of the ½-in. face joint Portland cement is also used. As has been stated previously on all parts of the up-stream face of the masonry, which are planned to be below the back-filling line, and which are formed of rubble masonry, care has been taken to secure well-shaped stones and to fill up the intervening joints very thoroughly with small stones or spawls. This is well shown in Fig. 2, Plate XI, where the joints are shown raked out and ready for the pointing which, as in case of the facing stone, is done with Portland cement. In this connection, particular attention may be called to the very great amount of refilling which is to be done on the up-stream side, particularly back of the main dam and above the limestone foundation where so much badly fissured rock was found, and which resulted, necessarily, in the great depth of the rock excavation. This refilling, together with the pointing of the up-stream face of the masonry which it covers, is expected to be effectual in stopping percolation through or under the dam, even if in the latter case small open fissures may exist. At any rate, if, as noted previously in the chapter on "Pumping," a head varying from 20 ft. on the up-stream side to 10 ft. on the down-stream side caused no appreciable variation in the pumping of an amount of water, which at that time might have equalled 3 500 000 galls. per day, it does not seem that the head due finally to a full basin can increase very materially such flow as may possibly have already taken place through the limestone foundation rock.

The contract price for the rock excavation is $1.95 per cubic yard. The amount excavated will slightly exceed 300 000 cu. yds. While the price is seemingly a liberal one, it must not be forgotten that, in the bottom work, the blasting and excavation were not done to ordered lines and grades excepting as so directed from day to day, as the only limit in depth was good rock when reached. This necessitated a very great amount of careful hand work, as well as slow and expensive work in finally getting the bottom ready for the masonry.

All the earth excavation work, which has amounted to nearly 1 100 000 cu. yds., was, under the specifications, let at one price, viz., $0.61 per cubic yard, to avoid complication, although, naturally, there would have been little difficulty in separating the amounts lying below river level, and involving pumping, from the portion remaining. The price was considered fairly low when the risks were taken into consid-

eration. The increase in the length of the main dam, which was determined upon in 1897, resulted in a considerable decrease in the maximum height of the embankment at its point of junction with the main dam. This change has also decreased the amount of embankment as originally planned, and the core-wall trench is now practically wholly in hardpan; the point of junction with the main dam having been advanced to the south until it found the hardpan overlying the bed rock.

The work of construction began in October, 1892; the contract having been let in the previous August. The time limit in the contract was seven years, bringing the date of completion to August, 1899. The extraordinary depths to which the rock excavation had to be made in order to secure a foundation for the main dam as well as the change made in the length of the main dam after the work was started, which involved quite an increase in the amount of masonry, justified an extension of time of perhaps one year. The dam, however, will hardly be finished before 1902, making the time of construction ten years instead of eight. This delay is largely due to the dilatory ways and methods adopted in the first two or three years of the construction work, as developments have shown clearly that the plans and methods proposed by the engineers for carrying on the work have so far provided for and anticipated all emergencies and contingencies, in kind if not in degree; nothing unforeseen having happened to materially delay or involve a change of plans beyond the increased depth of rock excavation found necessary and the increase in the length of the masonry dam, as previously mentioned.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

TEST OF A MECHANICAL FILTER.

Discussion.*

By EDMUND B. WESTON, M. Am. Soc. C. E.

Mr. Weston. EDMUND B. WESTON, M. Am. Soc. C. E. (by letter).—The writer has read Mr. Fuller's discussion with a great deal of interest.

The color data given in Table No. 3, from March 13th to 18th, inclusive (5 days), as originally furnished the writer, were incorrect. Instead of 0.30 for the raw water and 0.06 for the filtered water, the figures for the 5 days should read 0.50 for the raw water, and 0.10 for the filtered water.

The writer, of course, did not propose to do chemical work in regard to the subject in hand. As to chemical matters, he is aware that different chemists sometimes hold different views on a given subject.

The chemical work was referred to Professor John H. Appleton, of Brown University, a gentleman whose age, long experience and conservative judgment entitle his opinions to great weight; and the writer still feels entire confidence in the analytical work, and the opinions expressed by Professor Appleton in the chemical questions involved in this discussion.

Color of the Raw Water.—Mr. Fuller remarks that the paper "does not state the method by which the recorded results" on color were obtained. They were obtained by the platinum-cobalt method.

Mr. Fuller remarks that "in a pamphlet issued recently by the New York Filter Manufacturing Company * * * , it is noted that

* Continued from December, 1899, *Proceedings*. See September, 1899, *Proceedings*, for Paper, by Edmund B. Weston, M. Am. Soc. C. E., on this subject.

the color results were obtained by the Nesslerized ammonia scale." Mr. Weston.
Mr. Fuller misunderstands the pamphlet. The pamphlet states what "the unit of color" is; but does not state the "method." The platinum-cobalt method was used, but the unit of color of this method is, as the pamphlet states, "practically that color yielded by properly Nesslerizing 50 c. c. of water containing one-hundredth of a milligram of ammonia gas (or its equivalent)." It appears, therefore, that Mr. Fuller's inferences with respect to the color data are not in accord with the facts.

Relation between Color and Required Sulphate of Alumina.—It was desired, if the alkalinity of the water would permit, to obtain a bacterial removal of at least 99% without regard to the quantity of sulphate of alumina used. It was found that 1 grain of sulphate of alumina per gallon would do this without rendering the filtered water acid; and it was also found that when $\frac{3}{4}$ of a grain was used, the bacterial removal was less than 99 per cent. It was therefore decided to use 1 grain.

As to Mr. Fuller's inference that there was a considerable waste of chemical at times, Mr. Fuller probably intended the statement to be taken in a relative sense, as the whole amount of sulphate of alumina used daily was not considerable, averaging less than 28 lbs., the daily cost being about 46 cents.

It should be remembered that the filter was in practical service during the test, and was not being run as a laboratory experiment; and as the bacterial removal averaged more than 99%, and the color of the filtered water was hardly distinguishable from freshly distilled water, and was sufficiently alkaline to show that the quantity of sulphate of alumina was being kept within the proper limit, it would hardly have been practicable, even if a small quantity of the sulphate could have been saved, to have made, from time to time, minute changes in its amount, as the cost of the labor of doing so would have been of much more account than the cost of the sulphate of alumina which might have been saved. Then, if this refinement had been gone into, and an experienced person had been employed to have continually kept the run of the alkalinity of the water, the expense would have been many times greater than the cost of the whole of the sulphate of alumina used.

It would appear as though Mr. Fuller's inferences from his experience with the water of western rivers would not apply to river waters in the vicinity of Providence.

During the Providence filtration experiments, in 1893 and 1894, it was demonstrated that the percentage of color removed from the raw water could not be relied upon as a gauge, in respect to the removal of bacteria, and the results of the East Providence test show the same to be the case.

Mr. Weston. At East Providence, as has previously been stated, the paramount desire was to remove at least 99% of the bacteria from the raw water, provided that it could be done without exhausting the alkalinity of the water and causing the filtered water to be acid, the importance of the removal of the color from the raw water being regarded as secondary to that of the bacteria.

As the bacterial results for each day were not known until about five days afterward, on account of the time required for cultivation, it would not have been possible to have gauged accurately the quantity of sulphate of alumina, more or less, which might have been the most advantageous to have used each day; therefore, as experience had shown that ¾ of a grain of sulphate of alumina would not produce an average bacterial removal of 99%, and that 1 grain would accomplish the desired result, and as the filtered water was always alkaline when 1 grain was used, it was thought that the constant use of 1 grain per gallon was the most satisfactory method of applying the sulphate.

Alkalinity.—Mr. Fuller makes a considerale body of comments on the alkalinity of East Providence water. The writer discusses these comments briefly:

The alkalinity determinations were made as follows: two portions, each of 500 c. c., of the water were placed in flat white porcelain trays side by side. To each sample, 5 c. c. of solution of methyl orange was added. First one sample and then the other was titrated with standard sulphuric acid, the acid being so prepared that each cubic centimeter would neutralize 1 part per million of calcium carbonate in 500 c. c. of water. (The sulphuric acid solution was standardized by pure sodium carbonate; then its value in calcium carbonate was computed.)

Mr. Fuller objects to methyl orange as an indicator. The writer must rely on Professor Appleton's statement that methyl orange is, in fact, a sensitive indicator for acid and alkali, that it is widely used for this purpose, and is recommended by high authorities on water analysis. Indeed, it was used during the elaborate filtration experiments conducted under the direction of Allen Hazen, Assoc. M. Am. Soc. C. E., at Pittsburg, Pa.; and Mr. Hazen appears to have been entirely satisfied with the reliability of the alkalinity determinations made with methyl orange.

From certain experiments made elsewhere by Mr. Fuller, he forms the opinion that the filtered East Providence water, during a portion of the test, must necessarily have been acid. But this is an opinion. As the result of actual tests, the filtered water was alkaline. That is, a considerable quantity of the standard sulphuric acid was necessary to overcome its alkalinity.

Mr. Fuller states that, in his opinion, a filtered water might "contain several grains per gallon of undecomposed" sulphate of alumina,

and yet that such water might be slightly alkaline to methyl orange. Mr. Weston. In the East Providence filtered water there could not possibly have been several grains per gallon of undecomposed sulphate of alumina, since not more than 1 grain was added to the raw water.

Chemical Results.—Mr. Fuller appears to represent that the analytical determinations of alumina, Al_2O_3, in the raw and the filtered waters are of little account. The writer holds the opposite view. He thinks them of considerable importance. They certainly show that in all cases the amount of alumina, Al_2O_3, in the filtered water was very small. It varied from about 0.02 to about 0.06 of a grain per gallon. But the 1 grain of sulphate of alumina added in the coagulant contained 0.22 of a grain of alumina, Al_2O_3. It is plain, therefore, that at least a considerable part of the alumina, Al_2O_3, contained in the sulphate of alumina applied, was removed by the process of filtration. Then, again, the analytical determinations show that there was an average of 38% less alumina, in the filtered water, than in the raw water before the sulphate of alumina was added to it. The writer considers these interesting and important facts.

Bacterial Results.—The culture medium used was 10% gelatine, and the reaction was slightly alkaline. The bacteria were cultivated at the average refrigerator temperature, the temperature of the laboratory being high at all times.

Growths of Bacteria.—The writer fears that he did not make his possible solution sufficiently clear, and that Mr. Fuller has interpreted his intent rather too broadly. It occurred to the writer, upon three or four occasions when the number of the bacteria in the filtered water had increased in a much greater proportion than those in the raw water, that it might have been due to a few bacteria growing in the filter. It was not his intention, by any manner of means, to even suggest the inference that bacteria ordinarily propagate in mechanical filters as they do in slow sand filters.

Wash Water.—From records kept during March, April and May, while 1 grain of sulphate of alumina, containing about 22% of Al_2O_3, was being used, the average length of the runs of the filter, which is the period between washings, is shown to have been about 6.6 hours; the range being from about 5 to about 9 hours.

During these runs, the height of the surface of the water in the filter (which remains practically constant), was about 10.85 ft. above the surface of the water in the controller. The operating head, or the head consumed during the process of filtration, was about 10.25 ft., namely: the difference between the level of the water surface in the filter and the elevation of the water (corresponding to the head upon the inlet pipe of the controller), above the surface of the water in the controller at the time the filter was shut down for the purpose of being washed. Immediately after washing, at the commencement of

Mr. Weston. a run, about 2.92 ft. of the operating head was lost by friction, due to the water passing through the clean filter-bed, screens and outlet pipes.

On account of the desirability of supplying the raw water to the filter by gravity, the filter was made only 12 ft. high, which is 4 ft. less than the standard height of filters of the Jewell gravity type, of the capacity installed at East Providence. If the filter had been 16 ft. high, the standard height, the operating head would have been 4 ft. more than 10.25 ft., and, consequently, as the greater the operating head, other things being equal, the longer a filter will run, the average length of time between washings would have been longer than 6.6 hours.

As the filtered water is being pumped constantly from the filtered-water well into the mains, it would be inconvenient to measure the amount of wash water used each time the filter is washed. From several measurements which have been made, however, the indications are that the average quantity of wash water used does not exceed 4% of the total amount of water filtered.

Automatic Controller.—The operating head, during the test referred to by Mr. Fuller, was 9.35 ft., and the head upon the inlet pipe of the controller at the end of the test was equivalent to a height of 1.5 ft. above the water surface of the controller.

The preliminary tests of the class of controllers used at East Providence are made with heads ranging from 18 ft. above the surface of the water in the controller as a maximum, to 0.33 ft. as a minimum.

Cost of Operation.—The wash pump is driven by a water turbine wheel, of much greater power than is necessary, which had been installed for another purpose before the filter plant was contemplated, and as the East Providence Water Company owns the water privilege from which the water required to operate the turbine is derived, the writer hardly thinks it would be advisable for him to go to the expense of indicating the power required to drive the pump, although, as a matter of scientific interest, he would like to know what it is. He can state, however, that a test, made about two months ago, showed the maximum horse-power of the water pumped while washing the filter-bed to be about 14. The horse-power was computed by considering the maximum quantity of water pumped per minute, the water pressure at the discharge end of the pump, and the elevation of the pump above the water in the filtered-water well. If the filter had been 16 ft. high instead of 12 ft., the other conditions being equal, the horse-power would have been about 15.6.

The pumping engineer, who has charge of the filter plant, estimates that the cost of the labor required for taking care of it is about $0.50 per day. As the writer has already stated in the paper, no additional labor, other than that which was employed before the filter plant was

built, is required to take care of the filter plant. This $0.50, considering the present consumption of about 200 000 galls. would equal, proportionately, $2.50 per 1 000 000 galls. Of course the cost per 1 000 000 galls. would be proportionately reduced if the filter was running the entire 24 hours and delivering its full capacity of 506 000 galls., and it would be very much less per 1 000 000 galls. if the three other filters, for which the filter building was designed, were installed, and running at their full daily capacities. Mr. Weston.

In reply to Mr. Gould—the general practice in England appears to be, after the repeated scrapings of a filter-bed have reduced its depth to the minimum limit, to dig off the old sand in sections above the gravel and replace it with a layer of fresh or washed sand, the old sand then being filled in upon the clean sand.

The advantage of being able to sterilize the filter-beds of mechanical filters, the writer considers to be of much importance.

The writer appreciates highly the thorough manner in which Dr. Currier has treated the subject in his carefully prepared and instructive discussion.

All processes of filtration, to be successful, must have intelligent supervision. Professor Percy Frankland, whose connection with the London water companies is well known, states, in regard to slow sand filtration:

> "But the responsibility which we have seen attaches to this treatment of water cannot be exaggerated, for whilst when efficiently pursued it forms a most important barrier to the dissemination of disease germs, the slightest imperfection in its manipulation is a constant menace during any epidemic."

Professor William P. Mason, of Troy, N. Y., has stated, in regard to the subject:

> "A filter, of whatever type, is a more delicate piece of apparatus than is generally recognized, and it requires constant attention of the most careful kind. In the mechanical form of filter, this care must, of necessity, be constantly forthcoming, or the filter would not run a day; the English bed, on the other hand, may be, and to my knowledge is, at times grossly neglected, and that too where the volume of the supply would seem to call for more attentive supervision."

The samples of filtered water, to which Mr. Williams refers, were taken from the controller and not from the well.

It would seem, by the somewhat eccentric language used by Mr. Williams, that he has derived considerable satisfaction, in drawing from a small outline sketch some rather humorous inferences in regard to the construction of the filter building.

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PAPERS AND DISCUSSIONS.

THE GROINED ARCH AS A COVERING FOR RESERVOIRS AND SAND FILTERS: ITS STRENGTH AND VOLUME.

Discussion.*

By Leonard Metcalf, Assoc. M. Am. Soc. C. E.

Mr. Metcalf. Leonard Metcalf, Assoc. M. Am. Soc. C. E. (by letter).—Mr. Le Conte's statement that "there are many modern designs for masonry coverings, composed of concrete and iron tie-bars combined, which are equally durable, strong and efficient, take up less space inside and are cheaper and better in every way" is broad and sweeping. Expensive the groined arch masonry roofs doubtless are, though by no means always so when considered from a comparative point of view and the conditions they are called upon to meet. With the present prevailing high prices of iron and steel, for instance, they will be found more economical in many cases than steel and concrete construction. Freedom from the danger of corrosion, and cheapness of maintenance they certainly possess, together with strength and stability. Further study and experiment will doubtless determine the limits to which the dimensions of the arch may be safely reduced under different loads, with resulting economy of material.

As regards the actual expense, Mr. Hazen has stated that the cost of the Albany filter plant roof amounted to only $0.182 per square foot for the concrete masonry in place, and less than $0.28 per square foot, including the cost of piers, earth filling and seeding, manholes,

* Continued from October, 1899, *Proceedings*. See May, 1899, *Proceedings* for Paper, by Leonard Metcalf, Assoc. M. Am. Soc. C. E., on the subject.

entrances, fasteners, etc. In a small structure, where the centering cannot be used a second time, the cost is relatively greater. Thus, the centering alone of the Wellesley reservoir is stated by its engineer, Freeman C. Coffin, M. Am. Soc. C. E., to have cost $0.225 per square foot. The roof of the Concord, Mass., sewage storage well, of 57 ft. internal diameter and containing about 100 cu. yds. of masonry, designed by the writer, cost for Mr. Metcalf.

Centering	$0.18	per square	foot.
Concrete, materials	0.15	"	"
Labor and supervision	0.05	"	"
Total	0.38	"	"

Mr. Hutton has, unintentionally no doubt, misquoted the writer in saying "all the examples of the groined arch in engineering structures that have come to his knowledge are in the United States." What the writer said, was: "these examples, * * * limited as they are in number, are all in the United States that have thus far come" to his knowledge. The limits of the paper forbade reference to the many examples, to be found at home and abroad, of the use of the groined arch in ecclesiastical structures, and the comparatively few in engineering structures, to several of which Mr. Hutton has interestingly referred. One or two structures, in addition to those described by Mr. Hutton, which have come to the writer's notice in the course of his reading, are perhaps worthy of note. The groined roof-arches covering the filter-beds of the Zurich, Switzerland, water-works,* which are segmental, 14 ft. 9 ins. span, 4 ft. 1 in. rise, and 8 ins. thickness of concrete at crown; and those of the Berlin water-works reservoir at Charlottenburg, referred to in William Morris's paper on "Covered Service Reservoirs."†

In the United States at least two more groined arch reservoirs have been built in the past year, both for the storage of sewage—one by the Metropolitan Water Board of Boston, at Clinton, Mass., the other by the writer at Concord, Mass.

The investigations relating to groined arches by Mr. Hazen and Mr. Fuller, made in the course of the design and construction of the Albany filter plant, and the subsequent development of certain contraction cracks in that structure under changing temperature conditions, are most instructive and worthy of study. The writer is inclined to agree with Mr. Fuller that tension in the masonry over each pier, acting upon a principle "somewhat analogous to that of the dome," may be a factor in the strength of the arch; and, as Mr. Hazen has suggested, that the cantilever principle, as well as that of

* *Engineering News*, July 12th, 1894; and *Minutes of Proceedings*, Institution of Civil Engineers, Vol. cxi, 1892–93.

† *Minutes of Proceedings*, Institution of Civil Engineers, Vol. lxxiii, pages 1–60.

Mr. Metcalf. the beam and that of the arch, is called into the play. Just where one action begins and the other leaves off cannot be determined or demonstrated, but it seems very probable that tensile stresses are first called into play in the structure, and are followed by compressive stresses under which the arch finally fails, as was indicated so clearly by Mr. Fuller's experiments upon small models. This indicates that the proper place to introduce steel rods into the roof to strengthen the masonry is, not over the piers, but along the crown lines across which tension cracks first appeared, in the models tested, before the compression forces were called into play.

Mr. Fuller's method of computing the volume of masonry in any given arch is interesting. The work involved appears to be substantially the same as in the method pursued by the writer.

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PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE REACTION BREAKWATER AS APPLIED TO THE IMPROVEMENT OF OCEAN BARS.

Discussion.*

By Messrs. GEORGE Y. WISNER, J. FRANCIS LE BARON and GARDNER S. WILLIAMS.

GEORGE Y. WISNER, M. Am. Soc. C. E. (by letter).—The ideal conditions for obtaining satisfactory results from a single curved jetty are usually found where the structure can be made a continuation of the natural curve of the outlet of the harbor entrance. The necessity of placing the jetty on the north side of the entrance at Aransas Pass makes a reverse curve of the channel, with a tendency to shoal at the "Crossing," and in order to maintain the full channel depth of 20 ft. between curves without dredging will probably require a spur jetty on the south side of the channel as far out as the wreck *Mary*. Mr. Wisner.

Galveston Harbor, Cumberland Sound and Coatzacoalcos Harbor, Mexico, are good examples of natural conditions where a single curved jetty, properly located, would be certain to produce beneficial results. The Coatzacoalcas River is a silt-bearing stream, but the formation of the bar indicates that the load of sediment carried at times of floods is not sufficient to produce deposition from slight changes of velocity of current.

*Continued from November, 1899, *Proceedings*. See September, 1899, *Proceedings*, for paper by Lewis M. Haupt, M. Am. Soc. C. E., on this subject, and *Transactions*, Vol. xlii, p. 485.

Since the discussion on Professor Haupt's paper was prepared for press for Vol. xlii of *Transactions*, the discussions printed in this number of *Proceedings* have been received, and notice is hereby given that additional discussion on this subject will be collated and published in subsequent numbers of *Proceedings* and in the next volume of *Transactions*.

Mr. Wisner. The formation of the west side of the entrance is also such that a single jetty constructed on the east side of the channel would practically control the river current in a restricted channel across the bar into deep water, and if the littoral current is from the eastward, as has been stated, a single curved jetty would be the proper remedy to apply.

It should be stated, relative to the fourth conclusion on page 821* that the limitation of the use of single jetties at the mouths of silt-bearing rivers is intended to apply only to such streams as the Mississippi and Brazos Rivers, where the load of sediment during freshets is such that any diminution of velocity of flow produces deposition.

The phenomena observed at the South Pass jetties, relative to the effect of curves and of outlets through jetties near the shore, have an interesting bearing on some of the conclusions brought out in the present discussion.

The Act of Congress, under which the South Pass jetties were constructed, requires the maintenance of a channel through the jetties 26 ft. in depth, not less than 200 ft. in width at the bottom, and having through it a central depth of 30 ft. without regard to width.

In 1888, the conditions in the Pass became such that, in order to maintain the legal channel, dredging was necessary during most of the time when such work could be done. The writer was employed to make the necessary improvements to prevent the periodic shoaling and narrowing of the channel, and, from a careful study of the situation, concluded that the deposit of sediment in the channel was due to breaks in the jetties, allowing a large amount of water to escape before reaching the end of the jetties, and that the curvature of the channel was such that excessive depths developed near the concave jetty and caused sufficient deposit on the convex side to reduce the width at a depth of 26 ft. to less than 200 ft.

The construction of spurs, or short wing-dams, along the face of the concave jetty at intervals of about 500 ft., checked the tendency of the convex bank to encroach on the channel, and the repairs of the breaks through the jetty walls stopped the excessive deposits, and fully justified the conclusion that the correct remedy had been applied.

During the construction of the jetties at the mouth of the Brazos River, a heavy freshet occurred when the east jetty was built above high water for its entire length, and the west jetty only about three-fifths of its final length, which resulted in scouring out a channel 25 to 30 ft. deep between the jetties, and built up the bar beyond the outer end of the west jetty, so that the depth was less than previous to the flood. Careful study of the phenomena at both the South Pass and Brazos channels indicates clearly that, unless the flood waters of the rivers are confined within the channels until discharged into the

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littoral current outside of the bar, navigable entrances cannot be maintained. Mr. Wisner.

The amount of curvature given a channel fixes the width which can be maintained, and, if the curves be made sharp relatively to the volume of flow, sufficient width of channel cannot be maintained for the safe navigation of large vessels.

Experience shows that deep channels are maintained easily along the concave sides of curves, and are deeper and nearer the bank as the curve is sharper.

The steepness and character of the banks also have much to do in fixing the depth and position of the deepest water relative to both sides of the channel. A steep smooth bank along the concave side of a curve, with a shore having a gentle incline toward the bed of the channel on the convex side, is best adapted for maintaining the ebb flow parallel to the concave bank with minimum curvature of the channel.

The results thus far obtained by the incomplete works at Aransas Pass indicate that the design and location of the structure is well adapted for developing such conditions.

J. Francis Le Baron, M. Am. Soc. C. E. (by letter).—The writer has been familiar with Professor Haupt's theories in regard to breakwaters since 1888, and is glad that an opportunity has arisen for testing them under such favorable circumstances. The writer uses the word favorable, because it seems to him that the conditions at Aransas Pass are peculiarly adapted to successful improvement with a single jetty, although while the old Government jetty remained, the harbor was effectively bottled up. Mr. Le Baron.

We have here at Aransas Pass a large sandbank on the windward side of the channel, which is unquestionably caused by the diagonal action of the waves under the strong Northers moving the sand down before them, until they reach the outflowing current of the Pass.

If this current were not there, the sands moving continuously, ever as a resultant of the diagonal waves, would speedily extend the shoal to Mustang Island and perfect the littoral cordon. Anyone who is familiar with the sea must have noticed that this diagonal action is present on nearly all beaches the greater part of the time. The reason is that, with the wind blowing from all points of the seaward semicircle, except the normal, and, perhaps, perfectly parallel to the littoral, or even a couple of points off shore, there are some 19 points of the compass which produce a resultant diagonal wave action with the ground swell always rolling in from the outer sea, which is operative in this case, not only whenever the prevailing winds blow, but with every sea breeze, except that which is dead on.

The marked effect of strong winds blowing diagonally to a long straight coast line is seen on the Florida east coast between Cape Canaveral and St. John's Bar.

Mr. Le Baron. With a stiff Norther blowing, the surface water is blown bodily along the beach inside the breakers to the southward; the broken and lumpy water offering a better hold for the wind, which creates a strong race in the shallow water. This results in piling up the water to the south, and, to restore the equilibrium, a strong counter-current is established just outside the breakers; and so it happens that whenever a Norther is blowing there is always a stiff current running up the beach in the face of the wind, which is very noticeable. The writer has observed the same thing on the north coast of Honduras during easterly winds, which are the prevailing winds in that locality. He has measured the velocity there and found it to be 1.04 miles per hour inside the breakers. This current was heavily charged with sand which was being constantly hurried to the westward and deposited on the spit on the east side of the mouth of the Patuca River, which in this way had been built out 400 ft. in eight years above water. If this spit could be induced to extend itself some 2 700 ft. it would form a perfect jetty, and this is what the writer proposes to do; to assist Nature by building the lightest jetty possible along the extension of the spit, which lies below water, so as to prevent the water from the harbor overflowing the crest of the spit and the sand from windward driving into the harbor; the spit will then be protected in its work of completing the littoral cordon.

Now, this is what has happened at Aransas Pass, where conditions appear to be quite analogous with the work being done at Patuca Bar. The jetty built by Professor Haupt has arrested the drive of sand to the southward and has prevented the escape of water over the rim of the north bank. Its curved shape has undoubtedly assisted the cutting out of the channel, for the reasoning of Professor Haupt on this point is not only logical, but is borne out by facts as we find them in all natural water-courses having any current, where, as every truant school boy can tell you, the deepest water is found in the bends, where he goes fishing.

This is caused by the revolving motion of the water, caused by the current striking the concave bank at an acute angle, and being guided by it in a circular path causing the formation of whirlpools which suck up the sand of the bottom, in the same manner that everyone has noticed a miniature whirlwind on a hot day, suck up in its vortex the bits of paper and dust in the street.

Thus far, the writer agrees entirely with the author, but is forced to differ from him on several other points. One of these is the little weight he is disposed to give to physical surveys and examinations; and here is shown the fallacy of leaving a breach in his wall to admit the flood tide near the shore. While the writer agrees with the author fully in the great and well-nigh imperative importance of the study of comparative charts, he cannot conceive how any permanent improve-

ment of an ocean bar can be intelligently studied without the help of all the physical data which it is possible to obtain. These data, as the author observes, may sometimes seem conflicting and are often confusing, but it is the province of the engineer to so study and group them as to harmonize or eliminate the apparent discrepancies and contradictions, which, on careful scrutiny, will generally be found to arise from imperfect or improperly directed examinations, wrong locations of observing stations, or abnormal conditions arising from unusual phases of meteorological or fluvial regimen. Mr. Le Baron.

It also often happens, as in the work upon which the writer is now engaged, that no previous survey of the locality has ever been made.

If we are to depend only on comparative charts, an engineer could lay out a system of improvement without ever going on the ground in person, which the writer thinks few practical men would care to undertake; but instrumental surveys, which the author condemns, are only more careful and extended personal observations, elaborated too much, perhaps, at times, but still only a method of arriving at facts which a simple visit cannot fully determine. Unless we know the direction and intensity of all the forces operating on an ocean bar at different points, how are we to estimate the effect of these forces, after the changes produced by our proposed works?

The study of charts alone, without surveys to determine the intensity and direction of the forces operating, is also misleading, when studied in plan alone, as volume is apt to be overlooked.

It would seem that the author has fallen into this error when he says:

"The effect of the two jetties is to invert the natural trumpet-shaped opening, and to diminish the area of the gorge, which is transferred to the crest of the bar, thus reducing the tidal volumes, preventing the complete filling of the interior compartment," etc.

The author likens the natural river mouth to a trumpet. A more accurate comparison would be made by considering the trumpet to be flattened until it represented a cubical sector. This sector has a depth, let us say, of 4 or 5 ft. When we narrow the opening over the bar we simply turn the sector on its edge and thus gain in depth what we lose in width. The cross-sectional area is practically the same as before.

The author speaks of preventing the complete filling of the interior compartment of the Pass, by the construction of converging jetties. The writer would like to ask him if he has ever seen any river mouth or pass which has been improved by converging jetties, the interior compartment of which failed to fill. We know that this has been prophesied often, but the writer has yet to find a case where the interior compartment failed to fill. It was feared that something of this kind might happen on the St. John's River, Fla., where converging jetties have been built, and upon which the writer was

Mr. Le Baron. employed for several years as United States Assistant Engineer, but careful surveys and examinations, made by the United States Engineers after the jetties had been built, failed to show any diminution of volume.

It is admitted that jetties might be built so close together that almost no tide-water could enter, but in that case the river, or drainage, water from the surrounding water-shed would fill the compartment, which would be practically dammed up, with only a sluice-way between the jetties. This might happen while the works were in an unfinished state, before the jetty channel had acquired its contemplated normal depth, and the writer can see how we might easily raise the water level, but how we can lower it in the interior compartments he cannot see.

If we dredge and clear away all sand banks from the mouth of the river, or pass, and widen and deepen it, the low-water plane of the interior compartment may be lowered and the range of tide increased, but if we close up the mouth with a dam, which two jetties amount to, with a sluice-way between them, we will most certainly raise the water level in the interior compartments, and reduce the tidal range, just in proportion to the size of the sluice-way between the jetties. If, then, we raise the water level, we must have more water in the compartments, and the writer fails to see why fresh water is not just as good as salt water for navigation or for scouring.

Even where no river debouches through the jetties, and we have only a salt-water lagoon or harbor, the case would be rare indeed, where, taking a period of a year or several months, the effluent discharge did not exceed the influent. The reason for this is, that there is always a water-shed of greater or less extent to every harbor, and the writer ventures to say that Professor Haupt would never succeed in draining even a purely tidal basin by building a dam across it with an open sluice-way. Those who have had experience in draining tidal marshes know how difficult it is to accomplish this, even with tidal sluice-gates.

For these reasons the writer is utterly opposed to the plan of leaving an opening next the shore, or anywhere else, except in the jetty channel, for the admission of tide water. An opening through which the flood tide can enter permits more or less of the ebb tide to go out, and by just that much we lose scouring power in the ship channel, and are likely to set up a dangerous scour in the subsidiary channel, which may endanger the stability of the works, or bring an undesirable amount of sand into the harbor.

This problem is easily deduced by the *reductio ad absurdum*, for if a small channel is a good thing, a larger one is better. Then if we make it with the same cross-sectional area as our main channel, we will lose nearly if not quite the same amount of water, depending on

the relative velocities, and so the scouring effect is reduced; for, in spite of the incident, quoted by the author, of the flood last summer on the Brazos not deepening the channel between the jetties at the mouth, we know that lessening the volume and velocity of discharge produces bars and banks, which are swept away when floods come, and these cases are so well known and numerous as to require no demonstration. Mr. Le Baron.

If, then, instead of having two openings of say, the same width, with only 10 ft. depth each, we turn all the water through one of 20 ft. depth, the writer fails to see why the flood tide cannot find this opening and make as good use of it as is required. But the writer would prefer to keep all the flood tide out, if it were possible, and replace it with water of drainage or river water, even to the extent of creating a greater velocity between the jetties than might be desirable, which there would be some danger of doing if the mistake was made of making the channel too narrow.

This is a simple matter of computation, however, and it is the duty of the engineer to make it just right.

Leaving a jetty without any support, or with a gap between it and the shore, seems to the writer like sending a forlorn hope into an enemy's country unsupported. We are fighting the forces of Nature, and we must be careful and not be cut off from our reserves.

Another reason for leaving no gap in the line behind us is the ever-restless sand, which, as previously explained, is always traveling up or down the beach. If an opening is left here it deposits itself in the fair-way, but if the jetty is continuous to the shore the sand soon fills up the bight with a curved foreshore advanced to near the outer end of the jetty, backing and protecting the work, and rendering it indestructible by the waves.

For this reason, the jetty, in this position, can be very much less massive than would otherwise be necessary, and so the cost be reduced immensely and the river banks made continuous to the sea end.

This is what happened at the Suez Canal, Port Said; at St. Johns Bar, Fla., where the seaward angle of the south jetty filled up for over a quarter of a mile out; at Greytown jetty, Nicaragua; and what is sure to happen in every similar case on a sandy shore, where the prevailing winds blow along shore and into the bight made by the jetty.

The writer laid out the jetty at Greytown, making an angle of about 78° with the shore line to the eastward, from which quarter came the prevailing wind, and whence the sand was constantly moving along the beach. The location of this jetty was afterward moved quite a distance to the west, but the principle remains the same. In the writer's opinion, a large amount might have been saved in the cost of construction at Aransas Pass, if the jetty had been connected

Mr. Le Baron. with the shore in the first place, as, although longer, it could have been much less massive and more secure, for Nature would then have been assisted in building up the foreshore, which would have formed a solid spit on the back or windward side.

While the jetty at Aransas Pass has proved a success in securing deeper water, the writer does not believe that we can make a hard-and-fast rule, applicable to every bar. Each case must be studied with the help of all the data which can be obtained, and the success in this instance certainly does not warrant the author's sweeping condemnation of convergent or parallel jetties, which have been successful, as the author admits, at the Danube and Mississippi Rivers, at Tampico and also at St. John's River, Fla., Charleston, Newburyport, Galveston, Brazos River, the Swinemunde Haff, Germany, Volusia Bar, Lake George, and undoubtedly would have been at Cumberland Sound, which the writer laid out for General Gillmore, had it not been for the peculiar management of the work by Captain Carter.

At St. John's Bar the depth has been increased from 12 to 22 ft. At Volusia Bar, of which work the writer was in charge, first as engineer and later as contractor, the depth was increased from 4½ to 6 ft., all that was demanded by boats of the class navigating it. Many others might be mentioned.

Where dredging has been resorted to in connection with these jetties it has generally been done while the works were yet in an uncompleted state and to hasten the development of a deep channel at the instance of some impatient board of trade or meddling member of Congress. Sometimes it is rendered necessary, owing to the location of the jetties and the resulting jetty channel, as at St. John's Bar and at Greytown, where the new channel, in both cases, had to be cut through a sand bank 4 and 5 ft. above the water. At St. John's Bar the south jetty crossed the main ship channel, in order to make the resulting jetty channel take a more direct route to deep water outside the bar, and this channel had to be made through a large sand bank, known as Ward's Bank, which was dry at high water and nearly 6 ft. above low water; otherwise the north jetty would have had to be more than double its present length. It is a question if it would not have been cheaper, in the end, to have followed the natural channel, in this case, and assisted Nature to deepen it. After all, in all bar improvements, the main thing is the location of the jetties, and it is always safest to follow and assist Nature, when possible. This is shown, in the case under consideration, by the location of the old Government jetty, which was a flagrant example of wrong location, as it was built to leeward of the proposed channel, leaving it entirely unprotected from the encroaching sands to windward, with the result that it failed entirely to produce the desired results.

The use of curved, instead of straight, jetties is not new, as is well known. Here we have two jetties, both curved, but one properly located and the other improperly. The old Government jetty is evidently an attempt to follow the plan adopted at Swinemunde Haff, below Stettin, in Germany, where a curved jetty was built over 20 years ago, which has proved successful; but this jetty was built to windward of the channel, as it should be, whereas in the case at Aransas Pass the fatal mistake was made of building it to leeward. The fact that one jetty has been built here which has secured deep water does not prove that two jetties, if properly located, would not have produced the same, if not better, results; but if we can dispense with one of the jetties, and so save in cost, of course, it is preferable to do it. Mr. Le Baron.

The reason the author's jetty has succeeded and the Government jetty failed is, in the writer's opinion, entirely due to its location, and there is little doubt that a straight jetty in the same location would have produced the same results, as the main thing in this case was to protect the channel from the encroaching sands to windward. This done, Nature could safely be left to do the rest. In work of this kind we cannot cut out a pattern, lay it down on every chart of an ocean bar, and expect a jetty or breakwater built after it to open a channel, any more than we can take the locations, plans and profile of one railroad and use them for every other. Hardly any two cases can be treated exactly alike, but every one must be studied in detail, and with all the light of past experience and history to aid us in digesting and formulating all the facts obtainable in each particular case.

Gardner S. Williams, M. Am. Soc. C. E. (by letter).—Although the writer can lay no claims to experience in the construction of jetties or the improvement of harbors, it has come in his way to study quite closely the action of water upon the surfaces which confine it, and it seems to him that the so-called reaction breakwater is the only practical application of the real forces of erosion in moving water which has been cited in the present discussion. Without intending to in any way show disrespect for the older members of the profession or wishing to be accused of partisanship, it must be said that the true and most effective cause of scour by water currents seems to have been almost entirely overlooked, even by so eminent an engineer as Captain Eads. Mr. Williams.

The flow of water in straight channels of regular cross-section is not likely to be accompanied by strong scouring action, even at quite high velocities, because the direction of the flow of the individual filaments of the water is tangential to the bounding surface, but let a curve of long radius be introduced and something quite different will occur. The point or region of maximum velocity will be disturbed and carried toward the convex side of the stream, and the resulting rearrangement of the velocities will produce something approaching a spiral motion in the water, which will not be for any considerable

Mr. Williams. distance tangential to the bounding surfaces, and hence erosion will take place at once. If the curve be continued for a sufficient distance, the velocities rearrange themselves in conditions most closely conforming to their equilibrium and the scour diminishes, the direction of the currents becoming again tangential. If, now, a tangent be introduced, the arrangement of velocities in the filaments is again disturbed and excessive erosion again sets in, to disappear once more when they have again arranged themselves. The success of the reaction breakwater lies in the fact that it has set up these non-tangential currents, and the success of many double straight parallel jetties lies in the circumstances of their having received a stream from a tortuous river channel and forced it to straighten itself as it flows through them, thereby setting up the cross currents; while the failure of others of similar construction has been due to the circumstance that they received a stream already moving in comparatively straight lines, and have continued it in the same lines without setting up non-tangential currents. So, we should not expect that in the improvement of tidal harbors the passing of a volume of water in and out between straight parallel walls would produce any very great amount of scouring; but if it be passed along a curved surface, in and out, a continual setting up of eddy currents will be produced and the scouring accomplished. The writer, only a few days ago, had an opportunity of observing a current full of eddies moving stones half as large as a man's head, while a few hundred feet further on, the same water, with a lineal velocity twice as great, would not move stones the size of hens' eggs.

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RIVER HYDRAULICS.

Discussion.*

By A. MILLER TODD, Jun. Am. Soc. C. E.

A. MILLER TODD, Jun. Am. Soc. C. E. (by letter).—The author has contributed a valuable addition to the literature on hydraulics and on the subject of Mississippi River Improvement. Mr. Todd.

While the principles involved are not by any means new, the method of deduction, and of the transfer of the discharge-gauge relation from gauge to gauge is altogether so, as far as the writer knows.

The results obtained present at once, in convenient form, a discharge-gauge relation for each gauge considered, which is most interesting and useful to river engineers, especially those in charge of levees; for, if the relation is demonstrated to be reliable, it furnishes a clue to possible gauge heights when the river is confined, which, for any very great flood year, has never been done. In other words, say that we know the maximum discharge, due to pass a given place; to what height on the gauge will that discharge raise the surface level of the water?

The question of possible gauge height is a paramount one to the levee engineer when it becomes necessary to establish grades for his levees, when the river is confined throughout its length, or any portion of its length.

* This discussion (of the paper by James A. Seddon, M. Am. Soc. C. E., printed in the *Proceedings* for October, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to February 23d, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Todd. While the paper may not give conclusive evidence on the subject, yet we may apply the discharge-gauge relations, and compare results with those obtained by different methods of computation; which methods, in every case, are crude and more or less hypothetical. We may thus check former calculations, or inaugurate further investigations and calculations, in order to reconcile the discrepancy if any is found to exist.

It is very true that it is difficult and expensive to measure the discharge at a sufficient number of points, and to study very closely the progress and attendant phenomena and laws of water in its passage through great channels like the Mississippi below Cairo. Observations which have been made seem discordant and discrepant, and methods have been crude, but, beyond a doubt, work which has been done along this line cannot be praised too much. From the very first discharge measurements taken in the Mississippi, the methods have been improved upon, and the consequent results enhanced in value, up to the present time; and the writer thinks this improvement, both in methods and results, will continue. Therefore, he would not too hastily declare the author's method of studying river phenomena one which could supersede the old altogether; but, as before stated, the author's principles, properly applied, should furnish a valuable check and help to river engineers in future studies and investigations.

The author gives what he calls (average) "discharge scales." The writer has at hand a "Monograph" by the author "on Reservoirs and their Effects on the Floods of the Mississippi System" in which these "scales" * are plotted to a much larger scale, and they are very interesting to study. The writer has tested them with observed discharges taken at various times, and has found that the scales agree very closely with the actual results obtained from field discharge observations, and, in all instances and at all stages, they seem to show the average discharge.

In a number of lines of study, these average scales, as they are, should give correct results. But it occurs to the writer that, in many instances, before the levee engineer can accept the plotted values as correct, he must take into account the local condition of the water surface in the reach he has under consideration; whether it is rising, stationary or falling, and the rate of rise or fall; these are conditions which affect the flow. The author calls attention to the fact that, "the same change in the conditions of flow which would change the surface level by 1 ft. in a river from 50 to 100 ft. deep, would only change it by 0.1 ft. in a river from 5 to 10 ft. deep." So that, while the difference in discharge, due to whether the reach is rising or falling, is small at low stages, it becomes, at high stages, of such magni-

* House Document, No. 141; 2d Session, 55th Congress, Plates 8 and 9.

tude that it cannot be neglected. The author states that the time occupied by a change of level, passing through a given reach, is absolutely constant; but the writer understands that most, if not all, river engineers hold, from observation, that a fall travels more slowly than a rise; and, even at low stages, a rise in the reach, if the latter is long enough, will overtake a fall. Thus *m* in the author's formula cannot strictly be reckoned as a constant. Mr. Todd.

The author calls attention to a change of plane at Arkansas City, and offers possible explanations as to the cause, neglecting entirely what seems to be the true cause of the change; at any rate, as eminent an engineer as William Starling, M. Am. Soc. C. E., who is a recognized authority on river subjects, accounts for the identical change, and plots the identical figure used by the author, Fig. 7.

In a paper by Major Starling,* Fig. 24, on page 450, shows, evidently, that the change of plane is that which recurs regularly, shifting from one to the other, according to whether the river is rising, stationary or falling.

In that valuable paper Major Starling undertakes to foretell the probable gauge height of maximum flood discharge in reaches where the floods have never been confined. He frequently calls attention to, and takes into account, the difference in discharge, at equal stages, due to "rising river" and "river after rise," and he always obtains two curves, calling them the "two branches."†

In Fig. 16 the writer has plotted the discharge observations referred to by both the author and Major Starling, and has fitted and marked distinctively, curves through certain periods of rise and fall.

It will be seen that the fall on February 4th had something to do with the shifting of the discharge-gauge relation up to the stage on that date, and there can be plotted, through the points indicating a fall, a distinct and separate curve from that drawn through the points marking the rising stages. This second branch is traced down as long as the river falls. A rise commences on March 4th, and, instead of the discharge-gauge relation agreeing with the rise of December 30th to January 18th, the discharge seems to pass at considerably higher gauge readings. Under the circumstances, the writer thinks the foregoing fact should be expected. The average datum area was practically the same during both rises; while, from December 30th to January 18th, 19 days, the river rose 25 ft., or an average of 1.3 ft. per day, and from March 6th to March 23d, 17 days, it rose only 13 ft., or an average of 0.7 ft. per day.

For any portion of the reach in which Arkansas City is situated, if there are gauge relations, which cannot be doubted, then, for a rise of 1.3 ft. per day, the slope is bound to be greater than for a rise of only

* "The Discharge of the Mississippi River, *Transactions*, Am. Soc. C. E., Vol. xxxiv, p. 347.

† *Ibid*, p. 465, Fig. 29.

Mr. Todd. 0.7 ft. per day. Assuming, as the author does, that the miles per day traveled by a change of gauge height in a given reach *A B*, Fig. 17, is constant; assuming, also, that 1 ft. rise at *A* is equal to the same amount of rise at *B*, or that the gauge relation of *B* to *A* is 1, and that the time interval is 1 day; also, suppose that, on a given day, the gauge at *A* shows a rise of 0.7 ft., then the full line A_1-B will represent the general slope of the river on that day; the next day there will be 0.7 ft. rise at *B* and also at *A*, and for all succeeding days, as long as the increment of 0.7 ft. daily at *A* obtains, the slope on those days will continue parallel to A_1-B. If a 1.3-ft. rise is recorded at *A*, the dashed line A_2-B will represent the general slope for that day, and for all succeeding days of the 1.3-ft. rise. Now, for example, assume the distance, *A* to *B*, equal to 60 miles, then the increase of slope of the 1.3-ft. rise over the 0.7-ft. rise is 0.006 ft. per mile. The slope being a function of discharge, the discharge-gauge relation varies accordingly.

It is the writer's opinion, that, as long as the river is rising or falling at nearly a constant rate, if true discharge measurements could be obtained and plotted, the points would lie in two perfectly regular curves, one representing the rising river, and the other the falling river, for that rate of rise or fall; and points for any other rate of rise or fall will depart from these curves more or less. So that, for the conditions of the water surface, with constantly varying rates of rise and fall, the actual curve of discharge-gauge relations is never a regular one. But, between an imaginary curve, representing the discharges passed throughout a rise to extreme flood height, rising at the constant rate of the greatest known average rise per day, and a curve representing a fall under the same conditions, there may be constructed a mean curve which would give a basis by which to reckon discharge-gauge relations under all varying conditions of flow.

If the observations from which Fig. 16 was derived had been continued, and the river had fallen to a stage of 8 ft., without being influenced by any sudden rise or fall, the points obtained, the writer thinks, would lie about where the x marks are indicated. Taking these points into consideration, and also the points plotted at 50 to 51.5 ft., obtained from discharge observations of the Mississippi River Commission in 1897 and 1898,* and projecting a mean curve through all the points obtained, as indicated by the full line, this curve, in similar form, would correspond to the author's discharge scales, and is about what he would obtain for Arkansas City, if not exactly the same.

Fig. 16 shows plainly the departure of some of the observed discharges from the average discharge-curve. For investigations requiring the summation of discharge over a period covering a rise and the

* See corresponding reports of the Mississippi River Commission.

corresponding fall to about the same stage, the scales constructed by the author cannot be improved upon. But, as before stated, they will not do where the discharge covering any given day, or the maximum discharge, is wanted. To render the average discharge curve useful in such instances, corrections must be applied to all stations, at least below Cairo, in the Lower Mississippi; and, the writer thinks, that the correction depends principally on the slope of the surface of the Mr. Todd.

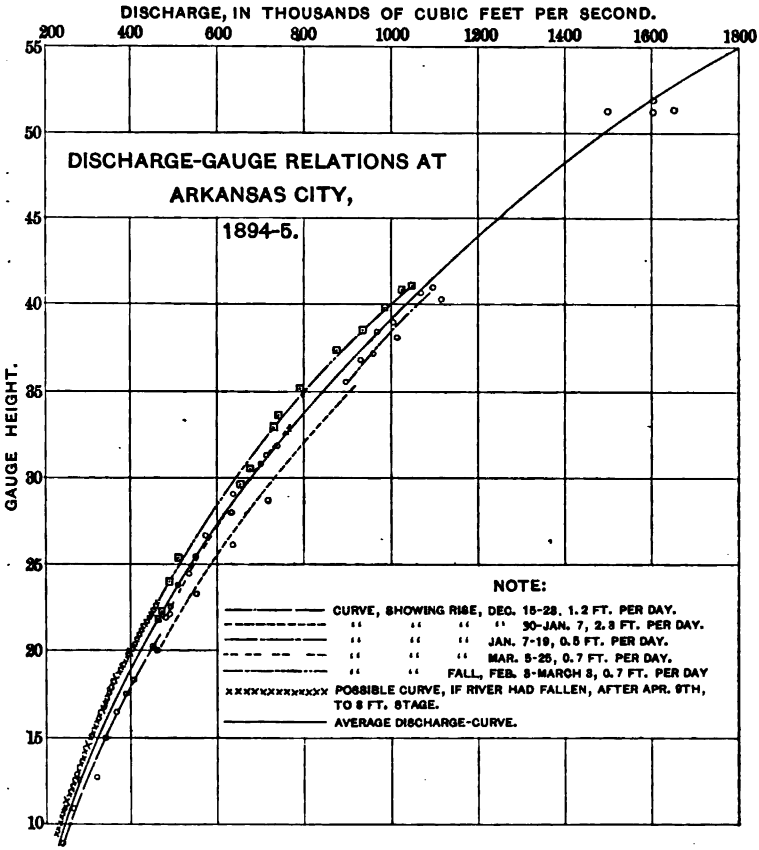

FIG. 16.

water in the reach in which the gauge station under consideration is located, and at the time the discharge corresponding to a given gauge height is desired. These slopes can be had roughly, at the present time, and, if the gauge stations were located and kept, as suggested by the author, the slope between them could be ascertained accurately at all times.

Mr. Todd. The author states that in certain instances corrections should be applied, and undoubtedly he has taken all the foregoing facts into consideration, and probably has a method of applying the correcction, not hinted at by him or conceived of by the writer. Possibly the writer, in his limited study of the problem, has laid too much stress on the slope as due to the many various conditions of the river, owing to the rise and fall alone; at any rate, he hopes that the author will state, in his closure, what his corrections are, and to what extent they will affect the quantities given by the discharge-scales.

There is one thing certain, as the author states, in transferring the investigations down the river below Cairo, the problem becomes complex indeed, due, principally, to the enormous number of variations and combinations of slope and momentum possible, according to the condition of the water level, not only in the single reach, but probably in several reaches above, which may affect the discharge materially; and also to the conditions and stage existing in the various tributaries. However, the only way to arrive at any tangible results, in clearing up and solving this great problem, is to keep hammering away at it; and the more we hammer, the sooner the desired end

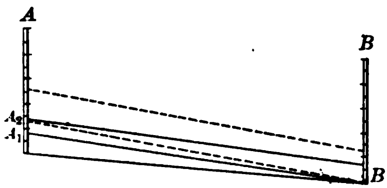

FIG. 17.

will be attained. After the levees have been maintained throughout several extreme high waters, we may expect to have some light on the subject. Heretofore, all the extreme high waters, after passing Cairo, have first been restrained in one place, and then have broken the levees and inundated basins in other places, the conditions of no one year repeating themselves the next; and it is impossible to study, with any degree of satisfaction, a flood spread over 100 000 square miles and upward. One flood, that of 1898, of comparatively short duration, which came within 2 ft. of the 1897 water at Cairo, was restrained successfully where leveed. The St. Francis levees were not completed, within some 100 miles of Helena, but the remaining portion is now being constructed. This basin being for a great part still open, the question of the effect of its closure on the flood plane, or gauge at Helena, is still problematical.

So that, for many years to come, the subject presented by the author will be an important study to all engineers interested in the Mississippi River problem, and it is to be hoped that he will give us, at some future date, the results of his further investigations.

MEMOIRS OF DECEASED MEMBERS.

NOTE.—Memoirs will be reproduced in the Volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ROBERT GILLHAM, M. Am. Soc. C. E.*

DIED MAY 19TH, 1899.

Robert Gillham was born in New York City, September 25th, 1854, being the third son of John and Clarissa Gillham. His father is an Englishman, but emigrated to America in early life, and has held important positions of trust under the Government of the United States.

His mother was an American, from one of the oldest and most respected families of New Jersey.

Having completed a course at a private school in Lodi, N. J., at the age of sixteen, he entered the Institute at Hackensack, N. J. He soon became assistant to Professor Williams, President of the Institute, and under his private instructions continued the study of engineering until 1874, when he began to practice his profession at Hackensack. Here, many problems were entrusted to him, the successful solution of which brought him much special work from New York City.

It was at this time that he was given the problem of utilizing the sulphur in zinc ores. After a most exhaustive investigation of all the methods in use, Mr. Gillham designed a furnace for desulphurizing zinc ores in such a way that, while the value of the zinc was in no way affected, the sulphuric acid gas was used in the manufacture of sulphuric acid. This was a most important step in the economy of zinc production.

Mr. Gillham removed to Kansas City, Mo., in 1878. He quickly saw the advantages of cable traction for the operation of street railways for that city, and at once proceeded to make plans for a cable railway to run from the Union Depot, up the precipitous bluff, direct to the business center of the city. The idea of such a thing was so novel, and the engineering difficulties were apparently so great, that his project was first regarded as chimerical by all those who were in a position to assist him. After many disappointments and discouragements, he succeeded finally in enlisting the necessary capital. His next problem was to obtain a franchise from the city. Here, he

*Memoir prepared by R. J. McCarty, M. P. Paret and G. W. McNulty, Members, Am. Soc. C. E.

encountered the resistance of the horse-railway company, which, notwithstanding its inadequacy, was being operated at a great profit. This resistance, owing to the great local influence of that company, proved, according to Mr. Gillham's own statement, one of the most formidable obstacles encountered in his career. However, with an unexampled tenacity of purpose, he persisted until success had crowned his efforts.

The spectacle of this young man of twenty-five, practically a stranger in a strange land, essaying to solve engineering problems which older heads had pronounced impracticable, proceeding with his task, undisturbed by the strictures of ignorance and undismayed by the immense power of municipal politics which he found arrayed against him, and conducting his enterprise to a pre-eminently successful issue, both in a physical and a financial sense, may justly be held up to all men, both young and old, for emulation.

The complete and astonishing success which finally attended his labors was emphasized, not only by the enrichment of his associates, but also by the adoption of his plans, to its immense profit, by the very company which had so bitterly opposed him.

It is sad to relate, however, that just in the hour of his triumph he was stricken down by an accident which caused him to hover between life and death for many months, when he should have been resting in peace from his labors and enjoying in health and comfort the feeling which comes from the consciousness of important services rendered and arduous duties well performed.

Having recovered his wonted health and spirits, he conceived the idea of an elevated railway over the low lands to the west of Kansas City, Mo., and across the Kansas River to Kansas City, Kans.

Owing to the then undeveloped condition of street-railway traction, he was forced to the use of steam, it not being practicable to use the cable. Having constructed the road west from the Union Depot in Kansas City, Mo., it soon became necessary to extend it eastward to the business center of that city. This called for a tunnel of 700 ft. under the bluff, on a 9% gradient, and made necessary the adoption of cable traction for the extension. The whole work was completed in 1887, but the road, from the necessity of having two kinds of power, and from the fact of its being a little in advance of requirements, did not prove at first a financial success.

About this time he also had charge of the construction of the Omaha Cable Railway, the Denver City Cable Railway, the Montague Street Cable Railway, in Brooklyn, and the Cleveland City Cable Railway, all of which enterprises were based upon the success of Mr. Gillham's Kansas City Cable Railway.

In 1888 he was retained to investigate the problem of changing the motive power of the Boston street railways from horse to cable.

Before he had completed his investigations, however, the practicability of electricity as a motive power for street railways had been demonstrated, and the Boston people decided to adopt that method of traction.

Shortly after this, in connection with the late John A. Wilson, M. Am. Soc. C. E., of Philadelphia, he made a report relating to an extensive elevated railway system for Boston.

He also visited Europe, and made extensive investigations of the problem of compressed air, in Paris and London.

In 1890 the Kansas City Elevated Railway Company had become involved in financial difficulties, and Mr. Gillham was asked to take charge of the rehabilitation of the property, and was appointed Receiver and General Manager of that company. He at once changed the motive power to electricity, and by his successful management and adroit and able conduct was able, in 1894, to effect a sale of the properties to the Metropolitan Street Railway Company, on terms highly advantageous to those he represented.

He was at the same time Receiver and General Manager of the North East Street Railway Company, of Kansas City, and conducted the affairs of that company to a highly satisfactory settlement.

Mr. Gillham's labors in connection with the receivership and management of the Kansas City Elevated Railway were exceedingly arduous, so much so, in fact, that in the summer of 1894, shortly after the settlement of that company's affairs, he was stricken with nervous prostration, and was compelled to spend some months in recuperation.

In 1895, he accepted the position of Chief Engineer of the Kansas City, Pittsburg and Gulf Railroad Company. At that time this railroad had only been built as far as Siloam Springs, Ark., 230 miles south of Kansas City. By the latter part of 1897, Mr. Gillham had the line completed and in operation through to Port Arthur, Tex., a total distance of nearly 800 miles from Kansas City, and had also built and in operation, about 90 miles of railroad in Missouri, north from Kansas City, and 50 or 60 miles of branch roads in the southern parts of the Pittsburg and Gulf Railroad System.

One great work which Mr. Gillham undertook, was the construction of the Port Arthur Ship Canal, which extended from Port Arthur, Tex., the southern terminus of the Pittsburg and Gulf Railroad, to deep water at Sabine Pass, Tex., on the Gulf of Mexico. This canal is 7½ miles long, 175 ft. wide and 25 ft. deep, and at the time of writing this memoir, but a few months after Mr. Gillham's death, is practically completed. The canal was begun in the latter part of 1896. The engineering difficulties were readily overcome by the foresight and resource of its Chief Engineer, though the opinions of several well-known engineers were against the practicability of the project.

From the first, the enterprise met with strenuous opposition from

adverse interests, and the opponents of the canal thoroughly exhausted every known method of resistance, both in and out of the courts. They even succeeded in getting the United States Government to interfere. Mr. Gillham was, however, not only an engineer, but a diplomatist of more than average ability, and having, moreover, a remarkable faculty of presenting his views and opinions in a plain and convincing manner, he soon broke down all opposition.

In 1896, he was appointed General Manager of the Kansas City, Pittsburg and Gulf Railroad, as well as of the Kansas City Suburban Belt Railroad.

In 1897, in addition to these positions, he also assumed the positions of General Manager and Chief Engineer of the Omaha and St. Louis and the Omaha, Kansas City and Eastern Railroad, and of the Kansas City and Northern Connecting Railroads.

On April 1st, 1899, he was appointed one of the Receivers of the Kansas City, Pittsburg and Gulf Railroad, retaining his position as General Manager both of the Kansas City, Pittsburg and Gulf and the Kansas City Suburban Belt Railroads, but resigning his position on the other roads mentioned.

On April 27th, 1899, as a result of the litigation of the affairs of the Kansas City, Pittsburg and Gulf Railroad, a change was made in the receivership of that company, but Mr. Gillham was, in recognition of his services and abilities, appointed by the Court as General Manager for the new receivers.

But this sphere of routine railroad work, though large, not being sufficient to satisfy his enterprising spirit, he at the same time sought other fields to fill the measure of his energy and talents.

Thus, at the time of his death, he held the following positions:

General Manager and Chief Engineer for the Receivers of the Kansas City, Pittsburg and Gulf Railroad; General Manager and Chief Engineer of the Kansas City Suburban Belt Railroad; General Manager and Chief Engineer of the Port Arthur Channel and Dock Company; President of the Armourdale Foundry Company; Vice-President of the Kansas City Elevated Railway Company; Director in the Missouri, Kansas and Texas Trust Company and the Kansas City, Pittsburg and Gulf Railroad.

He was also an active member of the Board of Park Commissioners of Kansas City, Mo., in which capacity he had done much to develop the park systems of that city.

He was a leading member of the Commercial Club of Kansas City, having, as such, devoted much of his time and energies to the work of that organization.

He was a member of the American Society of Civil Engineers, of the Society of Marine Architects and Naval Engineers, and of the Institution of Civil Engineers of England.

In December, 1881, he was married to Miss Minnie Marty, the daughter of a prominent capitalist of Kansas City. His wife, two daughters and an infant son survive him.

For some time before his death, the condition of his health admonished his friends that he was overworked, in fact during this period it would seem that nothing save his enterprising spirit and wonderful recuperative powers had borne him up.

On May 13th, 1899, having just returned from an exhausting tour of inspection over the Kansas City, Pittsburg & Gulf Railroad, he left his office at 6 P. M. as usual. As soon as he reached home he was stricken with a nervous chill, which was followed by pneumonia. His physicians stated that ordinarily he would have easily thrown off the disease, but that in his exhausted condition the worst might be feared. He lingered until 8.30 P. M., May 19th, when he passed peacefully away, surrounded by his family and nearest friends.

Mr. Gillham's interesting and valuable career resulted from a rare faculty to create his own opportunities, from an ability to use the same for the achievement of practical results, from an integrity and loftiness of purpose which directed his powers to the highest and most useful ends, from a genial and engaging manner, concerning the sincerity of which his contempt for all hypocrisy and his many deeds of disinterested kindness left no room for doubt, and from a courage, constancy and energy which left him ever undismayed.

It happened that just before his death he was selected to present a tribute to the memory of a prominent fellow citizen. This tribute contained the following words:

"Nature, having so constructed man that he might not exist without relation to his fellow man, has kindly placed among us some whose mission is to scatter peace and happiness all along the path of life. To every one of these there is a monument built of his own good deeds, and though inscriptions may not be upon it, God will know to whom it was erected."

These words will serve to show the standard of their author's excellence, and his life and deeds proclaim how well he followed it.

Mr. Gillham was elected a Member of the American Society of Civil Engineers, June 2d, 1886.

Vol. XXVI. FEBRUARY, 1900 No. 2.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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PAPERS AND DISCUSSIONS.

ILLUSTRATIONS.

The prices of publications are as follows: *Proceedings*, $6 per annum; *Transactions*, $10 per annum. Postage will be added when *Proceedings* are sent to foreign countries.

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ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 A.M. to 10 P.M.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

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Vol. XXVI. FEBRUARY, 1900. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

REPORT IN FULL OF THE FORTY-SEVENTH ANNUAL MEETING, JANUARY 17th and 18th, 1900.

Meeting called to order.

Wednesday, January 17th, 1900.—The meeting was called to order at 10.15 A. M., President Desmond FitzGerald in the chair; Charles Warren Hunt, Secretary.

The PRESIDENT.—Gentlemen, will you please come to order? Now, we have a long meeting and I hope you will all take an active part in it, so that we may get through by one o'clock.

Minutes of Last Meeting.

The first matter of business is the reading of the records of the last meeting, and Mr. Richardson moves that the reading of the records of the last meeting be dispensed with. Those in favor of that motion will signify it by saying Aye; opposed, No.

The motion was carried.

Tellers Appointed.

The PRESIDENT.—The first business is to appoint the Tellers, to canvass the ballot for officers for the ensuing year. The Chair will appoint Mr. T. McC. Leutzé and Mr. H. M. Rood, if those gentlemen will kindly begin the duties of their office.

The Secretary.—In the front room the Tellers will find the ballots prepared, and assistants to help open them.

The President.—As this is a long process of counting votes and will take some time, if any one here has not voted he will have abundant opportunity to do so; but the polls will be closed at twelve o'clock.

Report of the Board of Direction.

The first important business this morning is the reading of the report of the Board of Direction. The Secretary will please read it.

The Secretary read the report of the Board of Direction.*

The President.—Gentlemen, you have heard the report of the Board of Direction as read by the Secretary. What is your pleasure?

Mendes Cohen, Past President Am. Soc. C. E.—I move its acceptance, sir, and that it be placed on file.

The motion was seconded.

The President.—It is moved that the report be accepted and placed on file. Those in favor of the motion will please signify by saying Aye; opposed, No.

The motion was carried.

The President.—The next report is the report of the Treasurer.

The Secretary.—I do not see that the Treasurer is here.

The President.—Mr. Thomson is not here. Will the Secretary please read the report?

The Secretary read the report of the Treasurer.†

The President.—What is your pleasure with this report, gentlemen?

Mr. Cohen.—I move its acceptance and that it be placed on file.

The motion was seconded.

The President.—It is moved that the report of the Treasurer be accepted and placed on file. Those in favor of the motion will signify by saying Aye; opposed, No.

The motion was carried.

The President.—Does any gentleman wish to ask any question in regard to this report? Or, perhaps, we had better wait until they are all read. If that is the pleasure of the meeting we will wait until these reports are all read. The next is the report of the Secretary.

The Secretary read his report.‡

The Secretary.—I may say that the payment spoken of in the report of the Board of Direction of $10 000 has already been made. It was made on the 15th of this month.

The President.—Is that the whole of your report?

The Secretary.—Yes, sir.

The President.—What is your pleasure in regard to the report of

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* See *Proceedings*, Vol. xxvi, p. 7 (January, 1900).

† See *Proceedings*, Vol. xxvi, p. 13 (January, 1900).

‡ See *Proceedings*, Vol. xxvi, p. 14 (January, 1900).

the Secretary? It is moved that the report of the Secretary be accepted and placed on file. Those in favor of the motion will signify it by saying Aye; opposed, No.

The motion was carried.

Report of Committee on Award of Prizes.

The PRESIDENT.—The next business will be the report of the Committee to Recommend the Award of Prizes for the last year. Is the chairman of that Committee here, Mr. Secretary?

The SECRETARY.—Mr. Rodd is the chairman. I do not see him here.

The PRESIDENT.—Is there any member of the Committee here? Have you the report, Mr. Secretary?

The SECRETARY.—Yes, sir.

The Secretary read the following report of the Committee to Recommend the Award of Prizes and Medal:

Report of Committee to Recommend Award of Prizes.

PITTSBURG, PA., JANUARY 6TH, 1900.

To the Board of Direction,
AMERICAN SOCIETY OF CIVIL ENGINEERS,
220 West Fifty-seventh Street, New York City.

GENTLEMEN,—Your Committee to Award the Prizes and Medal for papers published during the year, ending with the month of July, 1899, begs to report as follows:

After careful consideration, we are unanimous in recommending the award of the—

Collingwood Prize:—To Paper No. 846, by Julius Kahn, on "Coal Hoists of the Calumet and Hecla Mining Company."

The Thomas Fitch Rowland Prize:—To Paper No. 836, by R. S. Buck, on the "Niagara Railway Arch."

The Norman Medal:—To Paper No. 850, by E. Herbert Stone, on "The Determination of Safe Working Stress for Railway Bridges of Wrought Iron and Steel."

Respectfully yours,
THOS. RODD,
Chairman.

The SECRETARY.—At a meeting of the Board of Direction held yesterday, it was resolved to award these prizes in accordance with the recommendations of this Committee.

The PRESIDENT.—Gentlemen, you have heard the report of this Committee. What is your pleasure? It is moved by Mr. French that this report be accepted and placed on file. Those in favor of that motion will signify by saying Aye; opposed, No.

The motion was carried.

The PRESIDENT.—I think, gentlemen, that this is the first time in the history of the Society that the award of the Norman Medal has been made to a member of the Society who is not living in this country,

Award of Prizes.

or is not an American. Mr. Stone, I think, is living in India, is he not?

The SECRETARY.—Yes, sir.

The PRESIDENT.—And that is something more than an ordinary compliment it seems to me, that this year the Norman Medal goes to a non-resident.

Report on Resolution Relative to Nominating Committee.

The following resolution was adopted at the last Annual Convention:

"That the Board of Direction be requested to consider the propriety of providing for the payment of the expenses of the members forming the Nominating Committee in attending the meeting of that Committee and report with further suggestions in regard to methods for the nomination of officers at the Annual Meeting."

Also it was moved, seconded and carried, that the Article of the Constitution referring to the matter of the Nominating Committee be referred to the Board of Direction for consideration and report.

A report of the Board of Direction has been made and has been adopted in regard to this subject. That report will now be read by the Secretary.

The Secretary read the report as follows:

Report of the Board of Direction in Matters Relating to the Nominating Committee.

The following resolutions were adopted at the Annual Convention:

"That the Board of Direction be requested to consider the propriety of providing for the payment of the expenses of the members of the Nominating Committee in attending the meetings of that Committee; and to report with further suggestions in regard to methods for the nomination of officers to the Annual Meeting" and

"That the matter of the revision of the Article of the Constitution relating to the nomination of officers be referred to the Board of Direction for consideration and report."

The Board respectfully reports that, after careful consideration of the whole matter, it does not seem necessary that any action be taken to provide for the payment of traveling expenses to members of the Nominating Committee, for the following reasons:

First.—The uncertainty of the amount involved, and the possible opening of the door to payment of other traveling expenses from the funds of the Society, for the attendance of Directors at monthly meetings, and for the attendance of members of special and standing committees.

Second.—The Nominating Committee, as at present constituted, consists of nineteen members annually, seven of which are elected each year and one being added each year by reason of his becoming a Past-President. Eleven members of this Committee therefore hold over and have more than a year in which, by correspondence, to arrive at the views of representatives of districts who might not be able to attend a meeting in New York. It is submitted that if the results of such a correspondence were communicated promptly to the new members of the Committee upon their election at the Convention, there would be

ample time to ascertain their views in regard to candidates before a meeting is held.

It appears to the Board, therefore, that it is strictly within the power of the Nominating Committee, as at present constituted, to so arrange its business that each member of it should have a vote whether he is able to attend a meeting or not, and that the intent of Article VII, Section 1 and 2, of the Constitution, in giving an equal representation on this Committee to each of the several districts, would in this way be carried out more thoroughly than by providing for the payment of traveling expenses. If, in order to have a voice in selecting nominees, members of this Committee must be present at a meeting, the element of time, which is probably as important as that of expense, is still in the way, and it seems doubtful if residents of the Pacific coast or Texas, whether their expenses were paid or not, would find it possible to leave their work for this purpose on a date previously fixed without consultation with them.

The Board recommends that no change be made in Article VII of the Constitution, believing that amendments to the Constitution are not desirable unless it is evident that they are imperatively needed, and that in this case such need has not been demonstrated.

By order of the Board of Direction,

CHAS. WARREN HUNT,
Secretary.

The PRESIDENT.—What is your pleasure, gentlemen, in regard to this report?

A MEMBER.—I move that it be accepted and adopted.

The PRESIDENT.—It is moved that the report be accepted and adopted by this meeting. Are you ready for the question? Those in favor of that motion will signify by saying Aye; opposed, No.

The motion was carried.

Proposed Amendment to the Constitution.

The SECRETARY.—Mr. President, in connection with this subject I have a letter from a Member of the Society, proposing an amendment to the Constitution on this subject and asking, although he knew that no action would be taken by the Society at this meeting, that the matter be brought before the meeting. It is from Mr. J. A. Ockerson, who was elected a member of the Nominating Committee last year.

ST. LOUIS, MO., DECEMBER 29TH, 1899.

MR. CHAS. W. HUNT,
Secretary Am. Soc. C. E.,
New York.

DEAR SIR,—I enclose proposed amendment to Sec. 2, Art. VII of the Constitution, duly signed by five members, as required.

I understand that it cannot be voted on at the Annual Meeting, but I would like very much to have it presented for informal discussion at that meeting.

There is an urgent demand for an amendment of this character, and I trust the object will be attained by its adoption. Personally, I would be satisfied if some result could be secured by the action of the Board of Direction.

Yours very truly,
J. A. OCKERSON.

Proposed Amendment to the Constitution (continued).

ST. LOUIS, MO., DECEMBER 21ST, 1899.

PROPOSED AMENDMENT OF SECTION 2, ARTICLE VII, OF THE CONSTITUTION OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

It has become apparent that the method now in vogue of selecting candidates for the various offices of the Society is open to serious objections and should be modified so as to eliminate these objections as far as practicable.

There seems to be an unwritten law which requires the Nominating Committee to meet in the Society House in New York when they select the nominees. The practical effect of this is to prevent the attendance of the members from remote districts, owing to the great expense of time and money involved, and the nominations are, therefore, left in the hands of a few who live nearby and can readily attend.

The importance of this matter is shown in the proposition made at the last Convention, that the Society should pay the expenses of the members of the Nominating Committee. This would impose a very heavy burden on the Society and one which is unnecessary.

By slight changes in the method of appointing the Nominating Committee, much of the difficulty can be obviated.

The method proposed calls for the selection of members of the Nominating Committee at the Annual Meeting. This can be done by sending out with the regular ballot for officers, a slip on which can be written each member's preference for member of the Nominating Committee.

From the names thus sent in, the Annual Meeting could probably ascertain the wishes of the members of the different Districts, quite as well as under present methods.

The Committee so selected should be required to meet at the time and place where the succeeding Annual Convention is held.

This would doubtless insure a much better attendance of the members of the Nominating Committee, would probably add something to the interest in the Convention, and would relieve the members of the unnecessary additional expense of attending two meetings where one would meet all the requirements.

For these reasons, the following proposed amendment to Section 2, Article VII, is submitted for the consideration of the Society.

J. A. OCKERSON,
R. E. MCMATH,
E. A. HERMANN,
W. S. LINCOLN,
J. F. HINCKLEY.

Section 2, Article VII, as it now stands:

At the Business Meeting of the Annual Convention of each year, seven Corporate Members, not officers of the Society, one from each of the geographical districts, shall be appointed by the meeting, to serve for two years; who, with the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society. The Board of Direction may prescribe the mode of procedure for appointing this Committee. This Committee shall present to the Board of Direction on or before the first day of October next ensuing, a list of nominees for the offices to be filled at the next Annual Election. The nominees shall be so chosen as to provide, with the officers holding over, a Vice-President and six Directors residing in District No. 1, and twelve Directors divided equally, with regard to

number and residence, among the remaining districts, Nos. 2, 3, 4, 5, 6 and 7.

Proposed Amendment to Section 2, Article VII.

At the Annual Meeting of each year, seven Corporate Members, not officers of the Society, one from each of the geographical districts, shall be appointed by the meeting to serve for two years; who, with the five living last Past-Presidents of the Society, shall be a committee to nominate officers for the Society.

The Board of Direction may prescribe the mode of procedure for appointing this Committee.

The Committee so appointed shall meet at the Annual Convention of the Society, and nominate candidates to fill the offices, named in Article V, so as to provide, with the officers holding over, a Vice-President and six Directors residing in District No. 1, and twelve Directors divided equally, with regard to number and residence, among the remaining districts, Nos. 2, 3, 4, 5, 6 and 7.

A list of nominees for the offices to be filled at the next Annual Election shall be presented by the Committee to the Board of Direction within ten days after the nominees have been selected.

Discussion on Proposed Amendment to the Constitution.

MENDES COHEN, M. Am. Soc. C. E.—Mr. President, I think that that amendment to the Constitution is very much in line with the necessities of the case. I am glad to hear it proposed. I was prepared to suggest something of precisely the same kind myself. I can see no reason why even more may not be done at the Annual Meeting, while the Nominating Committee may very well be appointed at the Annual Meeting. It appears to me that there is a much larger representation usually present at the Annual Meeting and prepared for business than at any other time in the year, and that this Committee might not only be appointed but might also convene, and. perhaps, make their nominations. But I am not clear in that regard, and propose no amendment to the amendment that has been presented. But I think there is one amendment that could be made to the amendment as presented. It stipulates that the Committee shall convene at the time and place of the Annual Convention. That time and place may be very inconvenient. I believe it is now proposed, if not fixed, that the Annual Convention of this year shall be held abroad. I think it might well be inconvenient for the Nominating Committee to meet and act there, and therefore the meeting place of the Nominating Committee should be at the place of the Annual Convention, or in the City of New York, as may be deemed most suitable or most convenient, within the views of the Board of Direction.

HENRY B. SEAMAN, M. Am. Soc. C. E.—Mr. President, I concur with the remarks of Mr. Cohen.

The PRESIDENT.—One moment, Mr. Seaman. I am going to ask every gentleman here to give his name as he rises, so that the reporter may get it correctly.

Mr. SEAMAN.—But I am very strongly inclined to nominations being made by ballot. In fact I think that is really the purpose of the present discussion. It is almost impossible to get any committee of

Discussion on Proposed Amendment to the Constitution (continued).

this Society, spread as it is in all parts of the country, together in one meeting to make the nominations. In fact the practice of the Nominating Committee has been not to permit votes by ballot. I think that practice should be corrected and that all work should be done by ballot, and in nominating this Committee six months earlier there will be plenty of time for the nominations to be made in that way.

PALMER C. RICKETTS, M. Am. Soc. C. E.—Mr. President, I think the general idea of such an amendment is a good one, but I do not think that the Board of Direction should have anything to do with fixing the meetings of the Nominating Committee. Very wisely, under the Constitution, these Boards, that is, the Nominating Committee and the Board of Direction, are continually existing and entirely separate bodies. The Nominating Committee creates the Board of Direction, and therefore I believe that the Board of Direction should have nothing to do with the meetings of the Nominating Committee. The same idea exactly could be carried out in an amendment to the Constitution, by fixing methods instead of leaving these methods to be fixed by the Board of Direction. Of course the idea is that the creature should not have the creator in his power.

The PRESIDENT.—Of course you know that we can do nothing about this matter here, to-day, but it seems to me that it is rather interesting that we should have an exchange of ideas on the subject. It may aid the consideration of the question farther along.

HENRY G. PROUT, M. Am. Soc. C. E.—Mr. President, if you are willing——

The PRESIDENT.—I think you have violated the rule, Colonel Prout.

Mr. PROUT.—Mr. Prout. If you are willing to have an exchange of ideas from all, and there is no motion before the house, I should be ready to exchange some of my ideas on the subject. It strikes me, sir, that the agitation is almost entirely without foundation. The practice is simply this, as Professor Ricketts says—the Nominating Committee determines its own method of procedure. It admits votes by proxy or by ballot, or not, as it chooses. That is entirely within the province of the Nominating Committee. The Nominating Committee, so far as I have ever been able to ascertain, gives all possible weight to the wishes of members of the Nominating Committee who are not present, and it is quite impossible for men scattered all over the country to vote by ballot with any efficiency in such a matter. Circumstances arise, conditions come up and change, such that a man whom a member in Los Angeles had in mind at the moment of sending his ballot, would not be a desirable man to vote for when it comes to the time of acting on the nomination. Therefore, it seems that to leave the matter as it is now, in the discretion of the Nominating Committee, is much the best way of managing it, and is most likely to result in getting the right sort of men into office. At least that is the result of my very

limited observation of the actual working out of this thing in practice. The fact is, Mr. President, that there is no such thing as running human affairs automatically. You cannot possibly make a constitution or set of by-laws or any code of procedure such that it will fit every case that arises. You have got to meet matters in such a way that human judgment can play when the time comes, and it always will be so. Therefore, the sum of my observation in this matter is that we are as well off now as we could be, in this matter of the Nominating Committee.

GEORGE B. FRANCIS, M. Am. Soc. C. E.—As Secretary of the Nominating Committee, last year, I endorse what Mr. Prout has said. The Committee gave full consideration to the views of every absent member, and he was practically considered present if he chose to acquaint the members of the Committee with his views. I think it would be utterly impossible for any general ballot of the Society to be taken on members of the Nominating Committee. The members have to be divided by districts, and I doubt if there is a member of the Society to-day who can tell the limits of every district. It is quite a difficult matter. It seems that the Nominating Committee represents the Society even before the Board of Direction, as regards the selection of its officers, and apparently the Constitution intends that the Nominating Committee, made up of a miscellaneous number of gentlemen, members of the Society, separate and apart from the New York contingent or the Board of Direction, shall have that matter in charge and shall not be subjected to such regulations as the Board of Direction shall make; and these gentlemen, I hardly think, could correspond with each other nineteen of them—in such a manner as to arrive at a list of officers to nominate them through the mails. It is almost necessary to get together, and as to the method of balloting, the practice last year was for the Nominating Committee to ballot and make each nomination that way, so that each had a voice in that matter, and I am of the opinion that nominating officers either ought to be kept out of the Board of Direction, as it is now, by the Nominating Committee, or else turned right deliberately into the Board of Direction. I think the present method of work is all right so far as I can see.

EDWARD W. HOWE, M. Am. Soc. C. E.—As former Secretary of the Nominating Committee I concur in the remarks of the last two speakers. I am willing to allow that possibly something might be done to give a little more time. The time is very limited in which to carry on the proceedings of the Nominating Committee. I should not have any objection to having the nominations for the Nominating Committee made at the Annual Meeting. I think if this matter of letter-ballot was adopted we would not get the best results. You know how easy it is for some one to write to a member of the Nominating Committee at a distance and ask him if he won't endorse a certain individual, who very likely is a very suitable man for that office, and

Discussion on Proposed Amendment to the Constitution (continued).

he, of course, is very willing to do so, and the Nominating Committee when they meet will get letters from different individuals suggesting certain nominations. When the matter is up for discussion reasons will appear why it is more desirable to nominate some other person. I remember one case in the Nominating Committee where a person had been suggested for an office. One member, a Past-President, nominated this person as a candidate, and subsequently, after discussion, another person was nominated, and that very man seconded this new nomination. Although he had already made a nomination of a gentleman for the office, he seconded the nomination made subsequently. It appeared to him, after discussion by the Nominating Committee, that it would be better to nominate the other gentleman, and I agree that the best results are obtained by discussion in the meetings of the Committee.

The PRESIDENT.—This is a pretty good debate, and I hope you will continue it. I think we have got about ten minutes more for this subject. I hope some of the younger men will take part in these annual meetings. Is there any further discussion?

HENRY MANLEY, M. Am. Soc. C. E.—I think that it is all right as it stands, provided they use that discretion which they are supposed to have *ex officio*. I happened to be a member the very first time this thing was tried under the present Constitution. The members, especially selected from districts at that time, took great pains to canvass their own constituencies to ascertain the desires and opinions of the members in those localities. I think the present plan is an excellent one and leaves the Committee entirely free. It is the primary meeting of the Society. Let them have their own way and do their work.

Report on the Acoustics of the Auditorium.

The PRESIDENT.—Are there any further ideas to be exchanged on this subject? If not, we will take up the next. Is Mr. Hering here? Mr. Hering, have you the report of the Committee on Acoustics?

RUDOLPH HERING, M. Am. Soc. C. E.—The Secretary has it.

The PRESIDENT.—Then, will you please read it, Mr. Secretary.

THE SECRETARY.—This is a report made by the Special Committee appointed by the Board of Direction to report to the Board, and has been adopted by the Board as a report to the Society:

To the Board of Direction,

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Your Committee, to which was referred the question of the improvement of the acoustics of the auditorium, after much study and careful consideration of the practical and theoretical phases of the question, concludes that the trouble is in no manner due to the design or conformation of the auditorium; that the breaking up of the ceiling surfaces by deep girders and the clere-story were in the interest of good acoustics.

The difficulty resulted from the fact that the auditorium was not sufficiently furnished. Carpets and curtains which have been added

have already greatly improved the acoustic properties of the auditorium, and your Committee recommends for the amelioration of the remaining slight reverberation, that the vertical panels on the walls be covered with burlap of such color as will harmonize with the rest of the auditorium.'

All of which is respectfully submitted.

RUDOLPH HERING,
GEO. A. JUST,
LOUIS DECOPPET BERG.

NEW YORK, JANUARY 9TH, 1900.

Mr. Hering, being present, might say something more about the work of the Committee.

The PRESIDENT.—Mr. Hering is expected to say a word on this subject.

Discussion on Acoustics of the Auditorium.

RUDOLPH HERING, M. Am. Soc. C. E.—I do not think it is necessary to say anything more excepting to mention the details of the inquiry. I might say that the literature of the subject was pretty well ransacked.

The PRESIDENT.—Mr. Hering, would you mind stepping forward into the middle of the hall? You are giving a practical demonstration of its poor acoustic properties. (Laughter.)

Mr. HERING.—The Committee first endeavored to find out what had been done elsewhere to overcome similar difficulties, and obtained the literature on the subject in America, Germany and France and studied it, and found that the question was one that could be scientifically treated. The main reason for the reverberation in this hall was very clearly the large extent of smooth hard surface, now remaining on the walls. A hard smooth surface reflects the sound as it would reflect a ball thrown against it. The angles of incidence and of reflection of the sound waves are equal, and the sound will travel about the room a great many times in one second. Sound travels about a thousand feet, roughly speaking, in a second, and consequently while the second syllable of a word is being spoken by a speaker the audience hears the first syllable yet in its reverberation. The syllables overlap each other, and this is the cause of the defect of which we complain here. The way to get rid of that defect is to cause an absorption of the sound waves, and that is done by having soft surfaces and stopping the reflection of the sound waves. It is also got rid of by a dispersion of the sound waves, and that dispersion is accomplished by these little ornaments. When the sound strikes a semi-spherical surface, it is thinned out, as it were, by reflection in various different directions, and consequently it loses its intensity. So that in Europe, in large halls, they rely almost wholly on the dispersion of the sound waves caused by a breaking up of the sides and ceilings. It seemed to us that the only thing left to do here is to soften up these panels, these hard smooth surfaces that are left; excepting, of course, the wall behind the President's desk, this should remain, if it were not for other reasons, so that the sound from the

Discussion on Acoustics of the Auditorium (continued).

speaker will be thrown out toward the audience, but all the other surfaces should be made as soft as practicable, and then you will have the absorption of the sound, the same as the carpet, which has already made quite a difference, absorbs it here.

P. A. PETERSON, M. Am. Soc. C. E.—Burlap is not a soft surface, is it?

Mr. HERING.—Yes, sir. The same material you find in the reading-room. There, you notice the sound is all absorbed when you talk. You cannot hear any reverberation at all.

Mr. PETERSON.—Burlap is a hard linen. When you paint it, it becomes almost as hard as wood.

Mr. HERING.—That is not what we meant—a material like that in the reading-room.

The SECRETARY.—I think the burlap put on the wall is dyed, not painted.

Mr. HERING.—Yes. The object is to absorb the sound waves just as a black surface absorbs light waves. Whatever will do that will accomplish what we desire.

J. N. GREENE, M. Am. Soc. C. E.—I would like to ask the gentleman a question. How do you propose to take care of the sound coming back at you from the broken ceiling? What does the Committee conclude on that?

Mr. HERING.—Well, we thought the ceiling was broken up enough. You see, the ornamentation on these deep girders is a good deal richer than it is on the sides, and we thought for the present that possibly treating the sides would be sufficient.

Mr. GREENE.—I thought the general impression was that the ceiling was broken up too much altogether now—too much echo and re-echo; isn't that so?

Mr. HERING.—That was not our opinion, sir. The more you break it up the more you disperse the sound waves, and the less reflection.

Mr. GREENE.—There is one thing sure. It is very difficult to hear in this hall. What the remedy is, we want to get at, I suppose, in some way, and as soon as possible.

Mr. HERING.—Well, the remedy, as the Committee suggests, is to increase the soft surfaces around it.

The PRESIDENT.—I wish to say that the Board of Direction has already taken the necessary steps to carry out the recommendation of the Committee. Really, there is nothing that could be done at this meeting about it unless the meeting sees fit to adopt some action about it that would conflict with the Board of Direction, but we have voted to go ahead and carry this out. I suppose the actual expense of putting burlap on the side-walls is very small. So, even if it is nothing but an experiment, it will not cost much.

FOSTER CROWELL, M. Am. Soc. C. E.—It seems to me that this

matter is entirely before the Board of Direction. It has been very interesting to hear the reasoning adopted by the Committee, because that is enlightening to us all. I should like to ask the Committee, or Mr. Hering, whether they considered it sufficient to put the burlap directly on the surface, or whether an intermediate thickness of porous substance—when I say porous, I mean porous—a soft substance, would be additional advantage, or whether he proposes to hang the burlap only in connection with the wall, and if so, how is it applied?

Mr. HERING.—The Committee did not go into detail. But I might say that various matters of treating walls have been adopted. I saw one that seemed to operate effectively in Worcester, Mass., in the City Hall, where the burlap was fastened on little slats about a quarter of an inch thick, nailed against the wall. The burlap was thus kept about a quarter of an inch away from the wall, which, of course, allowed still more softness, and the sound there, which was said to be so that you could hardly understand anybody, was completely absorbed, and now you can talk there—I was in the room myself—and be understood, and you do not get this reverberation which continues for about a second after the sound has been produced; and as a person generally speaks several syllables in a second, you see, those syllables overlap, and consequently it is not very clear.

Mr. PETERSON.—Have you considered the question of stringing wires across?

Mr. HERING.—Yes, sir.

The PRESIDENT.—I beg your pardon. A gentleman has the floor.

ROBERT CARTWRIGHT, M. Am. Soc. C. E.—I am very happy to corroborate Mr. Hering in what he has said, based upon a similar experience. I think when we first met in this hall, some of you remember that I suggested precisely the same thing, and said that we could get a premium to have it done, for I thought John Wanamaker would be glad to go to work and adorn the side wall for the purpose of advertising. I have had some experience in regard to covering walls in a new church costing hundreds of thousands of dollars, and having a tesselated floor, and we have had to go to work and do that very thing to accomplish the object. Now, it would be a minor expense to suspend around here some drilling and prove it at once, and when we make a permanent arrangement, do it with artistic taste, just as Mr. Hering proposes. Not only that, at the first meeting we had no carpet on the floor, and we got this reverberation; we have carpet on now, and it is very much better. The only objection I have to it is that somebody outside there is addressing a meeting while we are trying to hear our President, or the speaker in this room, and if we could only quash that, I think we would be much better satisfied with the proceedings of the meeting.

The PRESIDENT.—Gentlemen, this is all the time that we can give to this subject, unless some one desires particularly to be heard.

Report of Committee on Standard Time.

The next matter before the meeting is a letter received from Mr. Sandford Fleming in regard to Standard Time.

The Secretary read the following letter:

OTTAWA, JANUARY 13TH, 1900.

Mr. CHAS. WARREN HUNT,
Secretary, Am. Soc. C. E.,
New York, N. Y.

DEAR SIR,—I duly received your letter of December 18th, transmitting resolutions of the Board of Directors in the matter of Standard Time. I deferred a reply in the hope that I would be able to attend the annual meeting on January 17th, when I would have the advantage of consulting other members of the Special Committee with the view of submitting a joint report. As, however, I am afraid that I shall be unable to be in New York next week I deem it proper to write you. I feel that it is but respectful to the Board of Directors that I should acknowledge the receipt of the courteous communication which they have instructed you to send me, and for which, on behalf of the Committee, I desire in the fullest manner to express very cordial thanks.

The resolutions of the Board of Directors transmitted to me convey the information, *first*, that they have ordered to be discontinued the use of the 24-hour notation in the publications and correspondence of the Society; *second*, that they have appreciated the labours of the Special Committee on Standard Time, and that although the desired results have not all been realized, they offer an expression of thanks for the efforts which have been made by the committee during a long series of years.

You are good enough to mention in your letter that the present action of the Board "refers altogether to the question of the use of the 24-o'clock system by the Society." And that the Special Committee is invited to report as heretofore either jointly as a committee, or by myself as chairman.

Accepting the invitation of the Directors, I beg leave as Chairman of the Committee to submit the following explanations for presentation to the Annual Meeting to be held on the 17th instant.

It is necessary that I should revert to the early days of the movement in which the American Society of Civil Engineers has played so conspicuous a part. Twenty years ago the development of lines of transportation on this continent, and the circumstances of the age, demanded an investigation into the matter of time-reckoning. This Society appointed a Special Committee to deal with the question. The records of the Society set forth the action taken by the Committee, with the approval of, and under the instruction of, the Society. The course followed had an important influence in inducing Congress to assemble a conference of representatives of all civilized nations at Washington in 1884, and likewise in the deliberations of that conference; it led to the substitution of Standard Time for the old complex system, and thus in part furnished a solution of the evils which formerly prevailed, not alone on this continent, but also on other parts of the globe.

The principles of the reform in time-reckoning, long favoured by the American Society of Civil Engineers, have been recognized as unimpeachable by the highest scientific authorities throughout the world. In every feature, the reform has been found practicable. Its

essential principles, that is to say, the reckoning of time on the basis of a common standard, has been widely adopted by the nations, and Standard Time, so called, is now authoritatively introduced on each of the five continents. It is only with respect to a secondary feature of the reform, that is to say, the notation of the hours, that there has been hesitation in its adoption; and it can scarcely be held to be surprising that there is hesitation and delay in some quarters, seeing that the adoption of the 24-hour notation involves a departure from an old usage. The old usage, when looked into, may in this age be considered indefensible, but it is, nevertheless, one which we have inherited from past generations, and it is only by the slow process of education that it may be superseded.

Precisely the same difficulty was experienced in the reformation of the calendar three centuries ago. Members of the Society will be familiar with the history of that reform. It was promulgated in 1582, and while it was immediately effected in Southern Europe, hesitation to adopt the change was experienced elsewhere. It took eighteen years to introduce it in Scotland, and so successfully was it opposed by popular prejudice in favor of the old usage, that a hundred and seventy years elapsed before the reform was accepted in England; at length in 1752 it became the legal reckoning in that country. We all know that there still remains one powerful nation adhering to the old style of calendar which before 1582 prevailed everywhere. Although Russia has been so exceedingly conservative, there are causes in operation to-day, which I venture to think will lead that great Empire, not only to adopt the common calendar, but along with it the reforms advocated by the American Society of Civil Engineers, including that feature of Standard Time known as the 24-hour notation.

The 24-hour notation is no vain experiment. It is in use by astronomers all over the globe. It is in use for civil purposes in large parts of Asia and Europe. It has been tested for fourteen years, and it continues to be used, on the National Railways of Canada. Indeed, the evidence of facts establishes beyond all question that this plan of notation is favoured most highly by all those who have longest experienced its advantages. To my own mind it is perfectly obvious that the day is not far distant when no intelligent and progressive community will rest satisfied to remain without the benefit of this simple and rational plan of reckoning the hours of the day.

This being my firm conviction, perhaps I may be pardoned for pointing out that a mistake was committed at the beginning of the movement, when in the publications of this Society, the new notation was employed to the exclusion of the old familiar expression A. M. and P. M., etc. If, for example, it was desired to announce that a meeting of the Society would be held at half an hour after eight o'clock in the evening, it was not enough to employ the new notation and ignore the old. In my judgment both should have been given. Heretofore, in the circulars of the Society, only the new notation was furnished. To many this was unsatisfactory, and it is not surprising that steps should have been taken to discontinue the practice.

As our habits, good or bad, are matters of education, it would, in view of all the circumstances, be well at this stage to furnish the information intended to be conveyed, in both ways. Thus "A meeting of the Society will be held at the hour of 20.30 (8.30 o'clock P. M.)," or the announcement may be still further simplified by denoting the time or hour by a single letter (say H) used as a symbol, precisely

Report of Committee on Standard Time (concluded).

as $ is employed to denote money—thus "A meeting will be held at H 20.30 (8.30 o'clock P. M.)."

I will only add that the problem which this Society has done so much to solve during the past nineteen years is not yet fully mastered, and it will not be fully mastered until Standard Time, on the basis of the fifth resolution of the Washington Conference of 1884, be brought into general use. This will undoubtedly require the exercise of patience. But by the process of education it will inevitably be brought about; and it only requires to complete the movement, that the hours of the day be numbered in a single series from one to twenty-four.

It will be remembered that when railway men were asked by the Society some nine years ago for an expression of opinion on the subject, that more than 400 presidents, managers and superintendents, representing 140 000 miles of railway, were in favor of the 24-hour notation. This wonderful unanimity of opinion on the part of those controlling the great lines of transportation on this continent, established beyond all question that the new notation commends itself to men of the highest practical intelligence. The railway interests are not less interested than they were ten years ago, and to effect the desired change it is only necessary that they should arrive at some joint arrangement among themselves.

Important reforms of this character are not effected speedily. It is not easy to overcome the restraints imposed by prejudice and habit, which our ancestors have transmitted to us; it will, however, every year become less difficult as the process of education effects its work.

The American Society of Civil Engineers took a leading part in initiating Standard Time, and it has continuously stimulated the development of a great reform in time-reckoning, not on this continent alone, but throughout the world. It must be recognized to be desirable that the Society should participate in the complete fruition of the movement. At one time some were sanguine enough to think it possible that the final step would be effected on the opening of the coming new century, but whether then or later, I am satisfied that like the Gregorian reform, the modern time-reform must in the end become an accomplished fact.

Permit me again to thank the Board of Directors for their kind consideration. It has indeed been a great satisfaction to me to have endeavored to carry out their wishes, and to serve the Society as Chairman of the Special Committee for so many years.

Respectfully submitted,

SANDFORD FLEMING.

Discussion on Report of Committee on Standard Time.

The PRESIDENT.—Gentlemen, whatever other annual reports we have, we always have Standard Time with us, and it is sometimes the subject of prolific discussion. What is your pleasure with this report?

FAYETTE S. CURTIS, M. Am. Soc. C. E.—I move that the report be accepted and placed on file and the Committee discharged. (Seconded.)

The PRESIDENT.—You hear the motion, gentlemen. What is your pleasure on Mr. Curtis' motion? (Question called for.) Mr. Curtis' motion is that the report be accepted and the Committee discharged and the report placed on file.

FREDERICK S. ODELL, M. Am. Soc. C. E.—Mr. Chairman, I think that we all recognize the value of Mr. Fleming's services, and the services of the Committee of which he is chairman, and we know how much has been accomplished by that Committee. Mr. Fleming now makes a recommendation. I think it would be treating his report with some discourtesy simply to receive it and discharge the Committee. I believe that the 24-hour system of notation is yet to prevail, and I do not think that this Society should put itself on record as not favoring it. I should therefore object to that part of the motion which calls for the discharge of the Committee. I think that this Society will yet want to join hands with all the influences that are working for the adoption of a universal system of numbering the hours of the day from one to twenty-four. As Mr. Fleming says, it is simply a matter of education, and if we can educate ourselves or our successors to reckon the time from 1 to 24, instead of A. M. and P. M., I am sure it is a simpler method, although I am like the majority of the members of the Society—I never got used to 24 o'clock. I think it would be very desirable to adhere to that, and at the same time carry out Mr. Fleming's suggestion. I think it wise to simply receive this report and not to discharge the Committee.

The PRESIDENT.—The matter is open for discussion, gentlemen.

CHARLES A. MIXER, M. Am. Soc. C. E.—I am glad to stand with the last speaker. The 24-hour system has been in use by myself and on my work for the past five years. I find no difficulty whatever in having ordinary laboring men adopt it, and in all their records, which are continuous throughout the 24 hours, carry it out, and certainly with less error than if they had to use A. M. and P. M., and certainly the 24-hour system is more compact in making a record. It is entirely satisfactory to me, and I hope to continue it until it shall be universal.

Mr. MANLEY.—I move an amendment to the motion that the motion stand in this way: That the report be accepted and the Committee continued. (Seconded.)

The PRESIDENT.—It is moved that the report be accepted and the Committee continued. Does Mr. Curtis accept the amendment?

Mr CURTIS.—I do not.

The PRESIDENT.—The question is on Mr. Manley's amendment.

Mr. SEAMAN.—Perhaps the members present are not aware of the fact that the Society has formally dropped this notation since the last Convention. It is a matter that has been before the Society for several years in just this way; has taken up the greater part of Annual Meetings and Annual Conventions in discussing it, and I doubt very much whether any discourtesy to the Committee was intended. I do not think such was the case. It was simply that the time had passed when we should agitate this subject. The effort was made and the Society had decided that the matter should rest quietly, and I strongly

Discussion on Report of Committee on Standard Time (continued).

favor Mr. Curtis' motion, except that I would add, "discharged with thanks." We certainly have the most kindly feeling for the work that has been done by this Committee; but the purpose is to stop this everlasting agitation before the Society, the source of endless discussion year after year. I am in favor of the 24-hour system. I hope we will see it. But I hope we will not continue this agitation. I, therefore, oppose the amendment and would endorse Mr. Curtis' motion, adding, of course, with the thanks of the Society.

C. H. Graham, M. Am. Soc. C. E.—Mr. President, I am under the impression that at the last Annual Meeting a motion was made, I believe by Mr. O'Rourke, that the Committee on Standard Time be requested to present a final report. If my recollection is correct, it seems to me that the motion now presented is out of order and unnecessary.

The President.—I think the gentleman's memory is correct.

The Secretary.—At the last Annual Meeting the following resolution was adopted:

"That the Committee on Standard Time be requested to make a final report to the Society at the next Convention."

Mr. Curtis.—I think, Mr. President, my motion is in order. I understood they were to make a final report, and, in making a final report, I want to clinch it—I want to end it. We have had enough of this thing. We have had it at several meetings, and it has taken up more time than any other subject. Therefore, I think my motion is perfectly proper, that the Committee be either discontinued or discharged in connection with it.

Mr. Cohen.—Might it not be well to amend Mr. Curtis' resolution by the substitution of the following:

That the final report of the Committee be accepted and the thanks of the Society be conveyed to Mr. Fleming and the members of the Committee for their long-continued service. (Applause.)

The President.—Does Mr. Curtis accept the amendment?

Mr. Curtis.—I think I would accept that. I was intending after this to offer a motion of thanks. But as it is included all in one motion, I need not do that.

The President.—The question is, first, on Mr. Manley's amendment.

Mr. Manley.—If it is agreeable, I am entirely willing to withdraw that amendment, the fact being that the nuisance is already abated. The Board of Direction took action on that and stopped its use in the publications of the Society. I do not think the Society wants to take the position of stopping investigation or inquiry into a matter of such interest as the adoption of a system of standard time. But we have insisted for many years on putting ourselves in the wrong and making ourselves ridiculous. If Mr. Fleming and his distinguished

contemporaries desire to continue the subject, I hope the Society will give them an opportunity.

The PRESIDENT.—Mr. Manley withdraws his amendment. The question is now on Mr. Curtis' motion. Are you ready for the question as amended by Mr. Cohen? Those in favor will signify by saying Aye; opposed, No.

The motion was carried.

Report of Committee on Manipulation of Tests of Cement.

The PRESIDENT.—We now turn to the next business, Mr. Secretary. The Committee on the Proper Manipulation of Tests of Cement; Professor Swain, chairman.

Professor SWAIN read the report of this Committee.*

The PRESIDENT.—Gentlemen, you hear the report of the Committee on the Manipulation of Tests of Cement. What is your pleasure with regard to this report?

EDWARD P. NORTH, M. Am. Soc. C. E.—I move that it be accepted.

The PRESIDENT.—It is moved that the report be accepted and the committee continued. Are you ready for the motion? Those in favor of the motion will signify by saying Aye; opposed, No.

The motion was carried.

Invitation to Inspect Topographical Map.

The PRESIDENT.—I think there are some announcements to be made to the meeting in regard to the topographical map of New York. Is the gentleman present who is to do this? In the meantime if any gentleman has any business to bring before the meeting, now is an excellent opportunity to do it.

Mr. Secretary, have you the announcement about the topographical map?

The SECRETARY.—I have the following letter:

NEW YORK, JANUARY 15th, 1900.

To the Secretary of the American Society of Civil Engineers,
West Fifty-seventh Street.

DEAR SIR,—The topographical map of the City of New York, prepared in the Topographical Bureau of the Board of Public Improvements, for the International Exhibition in Paris, will be on exhibition in the Arion Hall, Fifty-ninth Street and Park Avenue, and believing that it might be of interest to some of the members of the Society of Civil Engineers to view the map, I send herewith fifty tickets of admission.

Respectfully,

L. A. RISSE,
Chf. Top. Engr. and Engr. of Concourse.
per FREDERICK GREIFFENBERG,
Principal Asst. Topogr. Engr.

It might be well to say that this map is 27 x 31 ft. in size, on a scale of 600 ft. to the inch, and covers an area of about 1 000 square miles.

* Professor Swain was not ready to hand in his report to be printed at the time of going to press, and it will therefore appear in a subsequent number of *Proceedings*.

These tickets, which Mr. Risse has kindly sent, are in the hands of the assistant to the Secretary, in the hall downstairs, and will be given to any of the members who may desire to go to view the map either to-day or to-morrow.

The PRESIDENT.—Are there any other announcements, Mr. Secretary, to be made?

Invitation to Inspect Belt Conveyor.

The SECRETARY.—Members interested in such matters as the following are invited by A. McC. Parker, M. Am. Soc. C. E., to visit at any time, during or after the meeting, his work at 38th Street and Second Avenue, to inspect the working of a rubber belt conveyor used in handling material which is being excavated from a large foundation and delivered on scows in the river. Mr. Parker is here and perhaps can say a few words as to just what the interesting part of the arrangement is.

A. McC. PARKER, M. Am. Soc. C. E.—Mr. President, the problem was presented to us there of getting out some 25 000 or 30 000 yds. of material very rapidly from a deep excavation. We installed, for that purpose, a rubber belt running on rollers, which were troughed so as to make the belt carry more than it would if flat. It was put up very hurriedly. It was driven by a 9 x 12 engine, and has been delivering material into scows lying at the foot of the property, at the rate of 450 or 500 yds. in five hours, at the rate of 100 yds. an hour; using 12 scraper teams to put it on the belt. I think it is something new in this part of the world, and that the application of such a contrivance is new in handling material in that line of work. Any member of the Society who would like to see it in operation is welcome to come there.

The PRESIDENT.—How wide is the belt?

Mr. PARKER.—Thirty-two inches. It runs over these troughed rollers or idlers, making the belt about 8 ins. in depth, about 12 or 14 ins. wide at the bottom, and then curving up on the sides. The material is delivered to the belt through 4 hoppers which are about 3 ft. above the top of the belt, making the whole arrangement about 4 ft. in depth. We dump into it as fast as we can drive the teams over the hopper, and there is no apparent limit to what the belt will carry off. It takes just as fast as we can drop the ½-yd. scrapers on to the belt. It takes anything at all you can get on to it. You can put on stones which take two men to lift which will go on at a speed of 400 ft. a minute, and mount up an incline of 26 degrees without any effort, and with no tendency to run down the incline. The belt has a total carry from the head pulley to the out-board pulley of about 225 ft. We have a square, 200 x 272, to excavate. Now, the bottom of the belt at the upper end of the lot is about 14 ft. below where the material is delivered, so as to shoot off and fill these broad dumping scows. We rise as we go up about 14 ft. But it does not make any difference

whether it is 14 ft. or 400—this grade of 26°—I believe you could roll eggs up it. It is a most amusing thing to see these round stones going up without any effort and without any tendency to come back.

The PRESIDENT.—I think if you keep on, Mr. Parker, you will have the whole Society there.

Mr. PARKER.—I do not believe they would want to leave it if they saw it. It is so remarkable to see these things—take a big stone that it takes two big men to lift, and you would think when it starts up that hill it would want to come back; but it does not; it rides up and that belt goes over the head pulley which is 4 ft. in diameter, shoots along and disappears. As you stand at the side of this 4-ft. pulley and look at it, the belt coming up at an angle of about 26°, you would naturally expect the material to fall down as soon as it gets up to the top of the hill. But it does not do that. It keeps on going. (Laughter.)

The PRESIDENT.—Mr. Parker, I think if you will stop there, they will believe your story. (Laughter.)

We have some more business to transact before the vote of the tellers is announced. Is there anything, Mr. Secretary, in regard to the Convention? That is an interesting subject. I know the members of the Society, from letters I have received, are all interested in that matter. Is there any announcement to be made about it? Annual Convention of 1900.

The SECRETARY.—Up to the present time nothing further as to the definite programme for the Convention in London has been arranged. The only thing that has been fixed is the time, which is to be the first week in July. It is very probable that the first meeting will be called on the Tuesday of that week, which is the 3d of July. But, further than that, the Committee has not made any plans. The matter is in the hands of a Committee appointed by the Board of Direction, with full power to make all the necessary arrangements. But Mr. Corthell was in Europe at the time the decision was arrived at, as to holding the Convention abroad, and as to the time of holding it, and he, at my request, very kindly, while in London made some inquiries and saw many of the members of the Institution. He is with us to-day and I think he would, perhaps, give us some information in regard to the feeling over there respecting the matter.

The PRESIDENT.—I am sure that I only voice the sentiment of the meeting in saying that we are extremely glad that Mr. Corthell is here, and I hope Mr. Corthell will respond and say a few words on this subject. We have, I think, six minutes and a quarter.

E. L. CORTHELL, M. Am. Soc. C. E.—Mr. President, then I have got to get right to business. I have been two months in Europe, but a great part of that was spent on this very matter, with a request from the Secretary that I would get some information on my return. In the first place, in reference to the Convention at London, I had a good opportunity of learning the wishes of the engineers, not only by per-

Annual Convention of 1900 (continued).

sonal conference with them, but also by personal correspondence with them. The Secretary of the Institution, Dr. Tudsbery, *Sir* Douglas Fox, the President, Mr. Webb, the Chief Engineer of the London and Northwestern, and several others, I saw and we talked the matter over. They understood it to be, and so did I, in an entirely informal manner. I was not an officer of the Society, nor was I a member of the Committee, but I thought I could bring back something from them that would be of interest and would be attractive to the members of this Society in their contemplated visit to Europe. I know what the personal views are of the two Past-Presidents of the Institution, *Sir* William H. Preece and *Sir* John Wolf Barry and of *Sir* Douglas Fox, the present President. They are very desirous that there should be a large representation of our members, and they are prepared to treat us very nicely indeed. to give us a cordial welcome, and not only that, but to arrange the programme of our stay of a week in London so that it will be greatly to our advantage and to our great pleasure. At the time I was there I expected, from the way the matter was going when I had left here, that there would be a large representation of this Society; perhaps in all, including the families of the members, five hundred people to go over there, and I hoped, and so did many of us at that time, that we could go in a body. On that supposition or assumption the members of the Institution are planning something, to be changed, of course, according to our movements, according to the number that may go, and other conditions. They wish us to arrive, if possible, together at Liverpool. They suggested that, because they wished to show us first, by an excursion that they will arrange, the Liverpool docks, the largest and finest in the world, as we all know, and the most expensive; then, by a steamer excursion, which will be arranged for, through the Manchester Ship Canal to Manchester, which will be a treat in itself. Then from Manchester by the London and Northwestern road to Crewe where Mr. Webb will show us, perhaps, the finest shops, I won't say in the world, but at least in Great Britain; thence to London. In London they wish to show us every attention, and are planning to give us a reception and *conversazione* in the Guild Hall of London, which they think they can arrange for. They also suggested that if we were there in large numbers and were most of us in one hotel which was our home while we were there—this was, perhaps, my own suggestion—that we could receive the British Engineers in that way, and so make it pleasant for them. I went to see our Ambassador, Mr. Choate, just the night before I was leaving and asked him if he would assist in such a reception at our hotel. He said to me: "Is that the Society, are those the gentlemen whose house I helped dedicate?" I said they are the very gentlemen. He said: "I will do anything I can for them. All they have to do is to say what they want me to do and I will be with them." (Applause.)

As to transportation, I did not know that that matter had been placed in the hands of transportation agents here who are perfectly able to carry out any programme for us; so I looked up the matter somewhat in regard to steamships, and particularly in regard to hotels, with the assistance or suggestions of Dr. Tudsbery and others. We can find accommodations probably all together, even if there are 300 or 400 of us, in a new hotel about a mile and a half away from the Institution where we are to hold our meeting; but very conveniently, right by there, are busses which can be called within fifteen minutes to take us back and forth. It is the Hotel Russell, on Russell Square, near Oxford Street. It is to be finished in March. I am awaiting now a letter in answer to my letter to the General Manager of the Frederick Hotel, who runs this new Great Central Hotel and others, in reference to this matter. In reference to the Institution, they wished to know how we conduct our Convention. I told them how we usually conducted it, and the meetings we desired to hold. They will give us the entire building, which you know is the finest engineering building in the world, will give us all facilities for our meeting, arrange the hall as they do for large occasions, which seats about 400 or 450 people, and it was suggested by *Sir* Douglas Fox that we should have a joint meeting of British engineers and American engineers to which they would like to invite members of all societies —any engineers that may be at that time in London, as he expressed it, so that we could talk to each other and tell each other how much we think of each other. He said he had a speech for the occasion himself. Many of those things are in their minds, and they are planning, and as soon as they know what we want and how many of us will be there, they will proceed to develop these plans, so that when we arrive the programme will be entirely arranged, and they will take charge of us, and give us attention on these lines, and any other lines that circumstances may indicate. How many minutes, Mr. President?

The President. —Six minutes and a half, Mr. Corthell.

Mr. Corthell.—Thank you. It is, I think, very important. I have sent out 150 letters to some of you gentlemen, most of you non-residents, within the last week and got some very nice replies, but they are not satisfactory to me, in reference to going. I have not overstated the attractiveness of this visit to Europe. It is going to be a visit that if we do not make it we will regret it all our lives, and if we do make it we will always be glad that we went, not only to see what we can in Great Britain, but in Paris. We are interested more or less in this or that Congress to be held there. Perhaps you may not know that when I left Paris, the principal delegate of the French Government, who has charge of the Congresses, told me that they have now reached a total of 125. We are not interested in all of them. But I have a memorandum here, copied this morning from the pro-

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grammes which I obtained in Paris of the following Congresses, and the dates on which they come, so that you may have them in your mind. The Mining Congress, mining and metallurgy, comes on June 18th and 23d. The Congress of Tests is from the 9th of July to the 16th, and the Congress of Navigation, in which many of us are interested, comes on the 28th of July and the 3d of August, and it was with that end in view I suggested that our Convention in London should be held in the week previous. This would be perfectly satisfactory to the Institution people, "because," as they say, "we are all here at that time; it is before our vacation—the Westminster Engineers—and Parliament itself is in session," so that it is the most convenient time to hold our convention in London. Then we can go to Paris, and those of us who are interested in the Congress of Tests of Material can immediately attend to that, as I know many are. Then with two weeks to see the Exposition and Paris, the Congress of Navigation comes off on the 28th of July, so that there will be six weeks of very interesting and useful experience for us to go through. As far as the Exposition is concerned, you know as much about it as I do, because it is in the papers, in the prints, every week. But I have given a good deal of attention to it. I have been over the Exposition grounds now for two years, several times, and I assure you it is going to be a very attractive exposition in many ways and in many new ways. An exhibition of the colonies of all the world is to be there, each one of which is going to be remarkably interesting, with the people themselves coming from those distant colonies and living as they live there. The new bridge to be built, which I described in a paper myself last February, is very nearly completed, and it will be something that bridge engineers will be very glad to see. It is probably the finest and widest street bridge in the world—140 meters. The buildings of the Exposition are nearly completed, and I think, with a few exceptions, they will all be ready when the Exposition opens. In the paper I read last February I showed a map on which was the Metropolitan Railway, 45 miles in all, but they are confining themselves now to the line underground, a tunnel or subway from the Bois de Vincennes to the Exposition, the Trocadero, with branches, so that the facilities for getting about Paris are going to be very convenient.

Another thing, I looked up the question as to the price of living there. The general impression——

The President.—Mr. Corthell, one moment. As it is now twelve o'clock I will declare the ballot closed. You will proceed, Mr. Corthell.

Mr. Corthell.—I made many inquiries in reference to it, knowing that it was a matter you would all be interested in, because it is going to be an expensive trip anyway. I think from all I can learn that the

increase in cost of living in Paris will be not much more than 20% of what it usually is. The people with whom I talked told me that they had to raise the price somewhat, because the price of everything that people used would be raised; consequently the *pensions*—and you know Paris is full of *pensions*—I don't mean a third-rate New York boarding house which we don't like very well, but the *pensions* in Paris, as a general thing, as far as I know, and I have lived in several of them, are very nice; the food is good, is well served, and they are well taken care of. Now, anyone can live in those *pensions* during the Exposition at about, comfortably, not over 10 francs a day, that is $2.

About transportation, what I want to do and what many wish to do is that we can finally get together a large enough body of our people to go together. It would be better in every respect.

The PRESIDENT.—You mean to go over together?

Mr. CORTHELL.—That is what I have had at heart and I cannot get rid of it. (Applause.) If we should go together we could be entertained better, we could be better taken care of, and it would please the people in Great Britain to have us come in that way, and I think from what I learn of our transportation people, in whose hands we placed this matter—Raymond & Whitcomb—that within a month or six weeks even from now, if we should find that we were really going in a body, we could cancel this arrangement we are now making with them on various steamers, and they will charter a steamer for us and we can go together. That is what I wish all of you would think about that are thinking about going, and follow up these letters that I have written to you and see if we cannot get together a good large body of American engineers going to Paris.

The PRESIDENT.—Would you mind my saying one word on that point while it is before the house? It seems to me that it is very important that we should know whether there are enough members of the Society and their families who will make a large enough party to go together. So far the Committee has had very little consolation on that point and very little information. Very few have responded. I want to emphasize that point. It will be necessary to have a response rather early, will it not, Mr. Corthell?

Mr. CORTHELL.—Yes, sir; certainly within a month, if possible.

The PRESIDENT.—Not many steamers are in service. A great many have been taken off and the others are rapidly filling.

Mr. CORTHELL.—Following that line of thought, I wrote to one of our friends, the Chief Engineer of the Liverpool Docks, knowing that he was right there where the two great English lines are, the White Star and the Cunard, to see what could be done. I have had some letters from the Cunard Line. They are nearly entirely booked on account of the disarrangement and the taking off of their vessels for

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the Transvaal war. They say that on our return there will be no trouble, if anyone has that in mind, about getting back to the United States in July or August, because people will not be returning in very large numbers, and the Cunard Company is of the opinion that there will be no trouble on returning.

Mr. Green asked me a question about the comparative expense whether we go separately or together. I think if we have a large enough body to go together, to take a special steamer, say, 500 people all told, that the cost would be less, I am not sure on that subject. The Committee may give a different opinion.

If I may be allowed just one moment about a matter which I have taken a great deal of interest in, as you know, and that is Navigation Congresses—I say Congresses advisedly. You may remember that at the last Convention I made a few remarks on the Brussels Congress, that was in May. Before I went to Europe in October I delivered—I guess that is the proper word—it took a trunk nearly to take my report down, which I spoke of at that time, delivered it to the Secretary of State. There will be about 250 pages of printed matter, and 115 illustrations on all subjects of navigation, rivers, canals, harbors, etc. While I have been away I have been informed by the Secretary of State that the report has been accepted and sent to Congress to be printed, and I just came from Washington. I spent twelve days there on the matter of these Congresses. I feel, perhaps, you have a right to listen to a word, because, I say, it is all for love; I have no interest in the matter except what you all have. The report is now in the hands of the public printer, and one object of my being in Washington these last twelve days was to arrange for the introduction of a concurrent resolution through Senator Platt, who is the chairman of the Printing Committee of the Senate, for a thousand extra copies, additional bound copies of the report. The Secretary of State in forwarding the report asked for no additional copies, consequently there will be only the regular number of 815 which go to several places and nobody ever sees them; that is, we will never see them unless there are those additional copies. Now, the Congress of 1900 coming this year I have had considerable to do with the matter, as all know, and it has been arranged that 2 500 copies of the programme be sent here, in French. I say in French for the reason that the French Government has decided that all programmes, all letters, everything relating to the Congresses in Paris this year shall be in French only. Now that has been translated, and you will all receive the programmes in a few days, with an English translation put into the fold, so that you will understand the whole matter, whether you know French or not.

Now, one more point and then I am done. I have been trying for a year, at the suggestion of prominent engineers in Europe, some of them members of the Society, like M. Pontzen; and, by the way, let

me interject a clause here—he is one of the leading engineers of Europe, and he is a member of this Society. He has been for many years Director of Routes of Navigation of the French Republic. When I told him that we were going to hold our Convention in London he was very happy. He said: "I am so glad, and I will be there myself with you," and these gentlemen, like Professor Timonoff, of St. Petersburg, who is a member of this Society also, and other leading men that I met in Brussels, favored this suggestion. I undertook, on my return a year ago, to see what could be done in holding any International Congress in the United States after the Paris Congress, and this last week there was a bill introduced, after considerable efforts to find a place for it—Senate 2 330—it will be introduced in the House, if it has not been already, authorizing the President of the United States to notify the International Navigation Congress, at its session in Paris this year, to designate the building which has been designated as "rooms in the Library of Congress," and either to appropriate $25 000 to pay the expenses of the occasion, or so much as may be needed of that sum in entertaining foreign delegates, and paying other expenses of the Congress. I am sending to-day from my office about forty of those bills to those of you who are interested in this subject, and I have sent for fifty more, and I will send them along. What I want you to do is to give the information to any that are not here to-day that anything that can be done in forwarding this matter in Congress by letters to your Congressman or your Senators will be in the line of bringing about something of great utility to the navigation interests of this country.

Now, just one more point. I have talked now nearly six minutes, I know. But it is of importance in connection with holding that Congress in the United States, that there should be a large delegation from the United States at the Paris Navigation Congress. Now, if they find on looking over the assembly over a thousand people from all parts of the world and only two or three from the United States, perhaps they will say, "we think you had better take more interest in the matter before we go to the United States." So now let it be understood. There may be several Government delegations.

The PRESIDENT.—A quarter of a minute more, Mr. Corthell.

Mr. CORTHELL.—But under the rules of the Congress there can be delegates from Chambers of Commerce, from Boards of Trade, accredited delegates as well as private members, and I wish you would all have that in view, and if you are going and are interested in the subject, to get somebody to make a delegate of you. (Applause.)

HENRY C. MEYER, F. Am. Soc. C. E.—I would like to ask a question—if it would be desirable for those members who contemplate being in Europe at the time of the Convention and yet find it more convenient to go earlier than the body will go, to arrange to meet in Liverpool, and if that were done, if the English Committee could then

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be informed about how many members of the American Society they would have to provide for. I think there will be quite a number that will want to go ahead of that time.

The PRESIDENT.—The Committee, I think, have already taken some action in regard to that.

Mr. MEYER.—These remarks of Mr. Corthell indicated that it was very desirable that all should go at one time, which would be the latter part of June, and that is why I made that suggestion.

Mr. CORTHELL.—May I say another word? I learned since I came back, that several are going earlier, as has now been mentioned by the last speaker, and I wrote a letter yesterday to a gentleman to please notify the Committee because he expects to go in May, and to be, on his return trip at the time of our meeting, and he will be there. It is all the same if we are there in London; the more the better, no matter whether they are going or coming.

The PRESIDENT.—Has the Secretary anything to say on the subject?

The SECRETARY.—In the matter of getting a steamer to take the Society in a body a great deal of effort was made by the Committee some time ago. Quite a number of the Trans-Atlantic companies were approached on the subject, and it was found to be absolutely impossible unless we had the funds with which to pay for the entire passenger accommodation on a steamer. Now, on the larger steamers that would amount to something like $30 000. The committee found also that we could get a slow steamer, a freight steamer, good-sized, perhaps 10 000 tons, on which temporary passenger accommodations for about 250 people are being placed for this year's business, and you could secure these accommodations on one of the regular trips of that steamer at a round rate, per person, of $50 for the outward passage. It seemed doubtful to the Committee whether the members of this Society would care to take that kind of accommodation without knowing what the service is going to be, or the food or anything else, except that the voyage will take about twelve days. But to get even that steamer with those accommodations would necessitate a deposit of $7 000. It was, therefore, impossible, the Society not having any funds to advance for such an enterprise, to engage any steamer before responses were received from the 250 odd members of the Society who had already said, in answer to the circular, that they would go to London, and it was hoped that after the last circular was issued in which a request was made that the Secretary be informed, as soon as arrangements were made by individuals, of their intention to go, and of the time they would probably go—it was hoped that there would be enough of them who had already made their arrangements, or had signified their intention of making their arrangements, to enable Raymond & Whitcomb, who are the agents in whose hands we tried to put as much of the business as possible, in order to keep it together, to enable them

to arrange, if possible, to charter a steamer for us. Now, that hope, as Mr. Corthell has suggested, is not entirely gone, but it is imperative in order that proper arrangements may be made for the Convention in London, that every member of the Society who decides to go, as soon as he so decides, should notify the Secretary of his intention, in order that our English friends may be informed of the number they may expect to meet over there.

ROBERT W. HUNT, M. Am. Soc. C. E.—May I ask whether the Committee or the Board of Direction of this Society have given any thought to the matter of combining with the other engineering societies? We are all aware of the fact, probably, that each one of our engineering societies has had identically the same invitation extended to it by the Institution, and they expect to have guests from all three. Now, many of us belong to all three and are confronted by the question of which invitation we are going to accept, if they are treated as three distinct ones. If we could charter a steamer and get enough acceptances would it not be desirable to repeat the history of the former journey abroad.

The PRESIDENT.—I think Mr. Clarke was present at the meeting last evening. You can answer that, can't you, Mr. Clarke?

Mr. CLARKE.—I beg pardon. I did not hear it.

The PRESIDENT.—Mr. Ricketts, can you give any information?

Mr. RICKETTS.—I was not able to be in town; so I cannot.

The PRESIDENT.—Mr. Manley, will you say a word on that subject?

Mr. MANLEY.—Mr. Chairman, I can only say a word. The Committee had a meeting last night at which they were informed that the representatives of one society had addressed a note to our Secretary saying that they were not going to make any arrangements to go themselves, but, perhaps, we would invite them to go with us. We considered that we could not invite members of other societies to go with us. But, at the same time, light is dawning upon me from the statement made that a general invitation has been issued by the Institution of Engineers to various bodies of engineers who propose to go there. I think that some of them—one body of mining engineers, if I have not forgotten—have already arranged to have their meeting some time in June. But I can answer the question in general terms by saying that no such general combination has been made. I might say further, that we are not able to go in a body ourselves, and that we don't know yet what to do, and that the arrangements thus far are very crude. The natural course of proceedings would be to appoint, on the part of the Society, a local committee, as we always do. We have a number of members resident in Europe, and that Committee, in connection with the Committee of the Society here, and with the Institution of Engineers, will be able to concoct a programme which can be intelligently carried out. But the Transvaal war seems to have knocked this to

Annual Convention of 1900 (continued).

pieces. Generally, as to our going there in a body, we have not yet been able to make such arrangements to take us over there to provide for our own, let alone other, people.

The PRESIDENT.—I am sorry to say that our time is all exhausted on this subject. Perhaps we will have an opportunity to take it up later. We will listen to the report of the Tellers, gentlemen.

Report of Tellers.

The SECRETARY read the report of the Tellers, as follows:

Report of the Tellers Appointed to Count the Vote for Officers.

NEW YORK, JANUARY 17TH, 1900.

MR. PRESIDENT:

We beg leave to submit the following report on the number of votes cast at the Annual Election:

Whole number of ballots received	570
Without signature	6
Valid ballots	564
For President:	
John Findley Wallace	557
F. P. Stearns	1
Elmer L. Corthell	2
Robert Moore	4
For Vice-Presidents:	
Rudolph Hering	560
Alfred Noble	561
George W. Rafter	1
John R. Freeman	1
Far Treasurer:	
J. M. Knap	564
For Directors:	
John F. O'Rourke	554
Henry B. Seaman	560
Thomas H. Johnson	564
Joseph Ramsey, Jr	564
H. B. Richardson	562
G. A. Quinlan	563
C. K. Bannister	1
W. F. Whittemore	1
William F. Merrill	1
Henry W. Brinckerhoff	1
Thomas J. Long	3

Very respectfully,

T. McC. LEUTZÉ,
H. M. ROOD,
Tellers.

The PRESIDENT.—Gentlemen, you have heard the report of the Tellers, and in accordance with that report I hereby announce that the Society has elected for the year ensuing, for President, John Findley Wallace; for Vice-Presidents, Rudolph Hering and Alfred Noble; for

Treasurer, Joseph M. Knap; for Directors, John F. O'Rourke, Henry B. Seaman, Thomas H. Johnson, Joseph Ramsey, Jr., Henry B. Richardson and George A. Quinlan. (Applause.)

Before introducing the new President, I should like to ask the Secretary if he has any announcements to make in regard to the programme?

Programme of Annual Meeting.

The SECRETARY.—Yes, sir. Perhaps it would be well to emphasize one or two points. The programme, so far as it is possible to state now, will be carried out as printed. Certainly, after the adjournment of this meeting, there will be a lunch downstairs, at or about one o'clock. At 3 P. M. Mr. Parsons' lecture will be given in this room, and inasmuch as the seating capacity of this room is limited, for sight of the pictures, to about 330, and as the ladies have also been invited to be present, I think it would be well for all who desire to hear Mr. Parsons to come early.

The reception to-night will take place as stated in the programme; and, weather permitting, the excursion to-morrow will be carried out as printed, visits being made to the two power stations, one of the Third Avenue Railroad Company, and the other of the Metropolitan Street Railway Company. There is every indication that we will be able to accomplish what is intended and go all the way around Manhattan Island. At 8.30 to-morrow night Mr. George B. Francis will address the Society on the South Terminal Station of Boston, which address will be illustrated with lantern-slides. I might say that it will aid the Committee of Arrangements, if those members who have not already secured tickets for the excursion to-morrow and for the reception this evening, will do so as soon as possible after the adjournment of this meeting. It is necessary to provide for quite a substantial lunch when you take a crowd off on the river for all day, and if we do not know how many we are going to have, somebody may go hungry.

Remarks by Past-President FitzGerald.

The PRESIDENT.—Before introducing the new President, I wish to say a few words in acknowledgment of the many acts of kindness received from the Society during the past year and in expression of my appreciation of the benefits derived from occupying this chair. Aside from the distinction which necessarily attaches to the office of President, there are many advantages which arise from a wide association with members, permitting an exchange of views on a more extended basis, or from a wider plane, than is possible as a member only. It has been a cause of regret to me that during the past six months important duties have called me away from my home and so far away that I have been unable to attend the meetings of the Society. This absence has, however, enabled me to exchange views and to come into close relation with engineers in sections of the country less familiar to me, and it has been a very gratifying experience. In Chicago there have been three occasions which have brought a large

Remarks by Past-President FitzGerald (continued).

number of engineers together which I remember with a great deal of pleasure. One was an excursion to examine the Yerkes telescope; another was a large dinner given in Chicago to Mr. Alfred Noble on the eve of his departure to Paris on duties connected with the Isthmian Commission; and the third was an excursion to Madison to witness the inauguration of Professor Johnson as Dean of the Engineering School of the University of Wisconsin. If any one supposes that enthusiastic excursions of engineers are limited to the East, he would have been very readily undeceived by attending either of these excursions. On the last occasion there were five cars laden with engineers and their families. All with whom I conversed had a high respect for this Society and its work, and an abiding faith in its future. It is perhaps true that a small current of pessimism may be sometimes observed; it is, however, confined, as a rule, to the older members of the profession who naturally miss familiar faces as age increases and who are less inclined to changes of method than younger men. It is true, I believe, that this Society was never in a better position for doing good work than it is to-day. It stands on a firm footing. The membership is rapidly increasing, and I believe that to-day it contains as large a proportion of able men as it ever did. The younger men have perhaps never been better fitted by training and association for carrying on the best work of the Society.

Our financial condition has been fully set forth in the Report of the Board of Direction, and you will see from that report how much the debt of the Society has been reduced and the work of the Society enlarged. We have many opportunities for the wise expenditure of our surplus. More time and money should be devoted to the Library. I have recently submitted to the Board a plan for extending its usefulness, and I hope the new management will take this matter in hand, and that the Society will approve of any judicious expenditures in this direction.

President Wallace introduced.

I now take pleasure in presenting Mr. Wallace, your new President. I feel sure that in him the Society has a wise counsellor and a loyal friend.

John Findley Wallace, M. Am. Soc. C. E.—Mr. President, and Gentlemen of the American Society of Civil Engineers: I thank you for the honor that you have conferred upon me, and I fully appreciate the fact that it is the highest honor that can be conferred upon an American engineer. Realizing this and my own unfitness I feel an embarrassment a great deal deeper than I can show, and which words cannot express. However, it has been the almost universal experience of American engineers to be called upon to fill positions and perform duties apparently beyond their abilities. I, therefore, accept this position, as I have many others of lesser importance, in the hope that a kind Providence may assist me in filling it, at least in such a manner as not to bring discredit upon our organization.

I had a few remarks prepared on the subject of the Annual Convention next year, but the ground has been so thoroughly covered by the remarks of Mr. Corthell and our Secretary and President and others, that I will only say that I would like again to impress upon you the importance of attendance at that meeting—the Annual Convention at London, during the first week in July. The time, place and circumstances surrounding that meeting will be such that every person connected with the Society should carefully consider the question of his personal attendance.

The importance of this meeting to the Society itself and the individual members thereof, I feel is so great that no one who can consistently and properly arrange his business affairs so as to admit of his attendance should fail to do so. This will be an occasion upon which the most important engineering organization of the new world will meet in the House of, and be the guests of, the most important engineering organization in the old world; an occasion upon which engineers of the two hemispheres will have an opportunity to meet, grasp each other by the hand, look each other in the eye, exchange views and ideas, and form personal acquaintanceships, the beneficial effects of which cannot be estimated.

With these few words I desire to remind you that economy is one of the fundamentals of our profession, and in striving for the highest economy we should, of course, avoid all unnecessary or uncalled for expenditure of time or energy. Following out this principle I must therefore ask you to excuse me from any further remarks at this time, as, while our Constitution specifically provides that the President of the Society shall formally address the members at the Annual Convention, it does not require any speech from the newly elected President on this occasion. The wisdom of this I have never so thoroughly appreciated as at this moment. I again thank you, gentlemen, for the confidence you have reposed in me. (Applause.)

President Wallace then took the chair.

The President.—I wish to announce, gentlemen, that it has always been the habit of engineers to work before they eat, and I, therefore, call the attention of the Board of Direction to the fact that we will have our meeting immediately after the adjournment and before lunch, in the Secretary's office on the first floor.

Mr. FitzGerald.—Mr. President, if you will allow a suggestion. If we have any time before we proceed with the important function of eating, I might suggest that we continue the discussion of the matter of the Convention, if the meeting feels so inclined and if the lunch is not ready.

The Secretary.—Mr. President, it is, of course, for the meeting to decide this question; but at three o'clock in this room there will be a lantern-slide exhibition. I think, perhaps, I might call attention to

the fact that usually, in the day time, we do not have to use these electric side-lights, but it has been necessary to shut out the daylight from the side windows, and it was a little dark this morning, so we turned on the side lights. But to transform this room into a dark-room is quite a trick, and as much has still to be done the Secretary would be glad if this meeting would adjourn as soon as possible. (Applause.)

The PRESIDENT.—What is your pleasure, gentlemen?

Mr. FOSTER CROWELL.—I move that we adjourn.

The motion was carried and the meeting adjourned.

Wednesday, January 17th, 1900.—The meeting was called to order at 8 P. M. President John F. Wallace in the chair; Charles Warren Hunt, Secretary; and present, also, about 450 members and guests, many of whom were ladies.

William Barclay Parsons, M. Am. Soc. C. E., delivered a lecture, illustrated by the stereopticon, describing a survey in China recently made by him, and giving an account of his experiences, and of life as found on a journey of 1 100 miles, 500 of which were through the unexplored province of Hunan, the most anti-foreign section of the Chinese Empire.

At the conclusion of the lecture a vote of thanks to Mr. Parsons was passed unanimously.

Adjourned.

Thursday, January 18th, 1900.—The meeting was called to order at 8.30 P. M., Past-President Mendes Cohen in the chair; Charles Warren Hunt, Secretary; and present, also, 138 members and 22 guests.

George B. Francis, M. Am. Soc. C. E., presented a paper entitled, "The South Terminal Station, Boston, Mass." The paper was illustrated with the stereopticon.

Owing to the lateness of the hour, written discussions from Messrs. J. R. Worcester and Herman Conrow were not read.

Adjourned.

EXCURSIONS AND ENTERTAINMENTS AT THE FORTY-SEVENTH ANNUAL MEETING.

Wednesday, January 17th, 1900.—After the business meeting lunch was served at 1.30 P. M. in the Lounging Room, and at 3 P. M. a large audience assembled to hear Mr. Parsons' lecture on China.

At 9 P. M. a Reception was held in the Society House, and was largely attended.

Thursday, January 18th, 1900.—At 10.30 A. M. the steamer *Valley Girl*, with about 300 members and guests, including many ladies, left the pier of the Department of Docks, at the foot of West Fifty-seventh Street, for an excursion around Manhattan Island. The steamer proceeded up the North River to Spuyten Duyvil Creek, where the new double-track drawbridge of the New York Central and Hudson River Railroad is being constructed. Through the kindness of W. J. Wilgus, M. Am. Soc. C. E., Chief Engineer of the N. Y. C. & H. R. R. R., a set of drawings of the bridge was placed on view on the steamer, and leaflets, descriptive of the bridge, were distributed.

Passing through Spuyten Duyvil Creek and the Harlem Ship Canal to the Harlem River, the first stop was made at Two Hundred and Eighteenth Street. Here the foundation of the new power station for the Third Avenue Railroad Company was inspected.

Resuming the trip down the Harlem River, the new "Speedway," the Washington Bridge, High Bridge and the numerous drawbridges spanning the river were viewed from the deck of the steamer. Lunch was served *en route*, and the steamer passed down the Harlem and into the East River.

By the courtesy of H. H. Vreeland, Esq., President, and M. G. Starrett, M. Am. Soc. C. E., Chief Engineer of the Metropolitan Street Railway Company, a visit was made to the company's new power station at the foot of East Ninety-sixth Street.

The steamer then passed down the East River, and, after making a stop at the Recreation Pier, at the foot of Twenty-fourth Street, to allow some of the excursionists to land, proceeded to and around the Battery and up the North River to Fifty-seventh Street, thus completing the circuit of the Island.

At 8.30 P. M., George B. Francis, M. Am. Soc. C. E., presented a paper, entitled "The South Terminal Station, Boston, Mass." The paper was illustrated with stereopticon views.

After the lecture there was an informal "Smoker," which was enjoyed by about 200 members and guests.

The following list contains the names of 350 members of various grades, in attendance at the Annual Meeting, Lectures, Excursion and Reception. The list is probably incomplete, on account of the failure

of a number of members to register, and does not include the names of any of the guests of the Society or of individual members.

Aiken, W. A.....Harrisburg, Pa.
Allen, Calvin H...New York City
Aycrigg, Wm. A....Omaha, Neb.

Bacon, John W..Danbury, Conn.
Bailey, George I....Albany, N.Y.
Baldwin, Fred. H..Bayonne, N. J.
Baldwin, Wm. J...New York City
Ballou, G. L..East Berlin, Conn.
Bascome, W. R..Brooklyn, N. Y.
Bauer, J. L.......New York City
Baum, George....New York City
Beahan, Willard......Easton, Pa.
Belknap, W. E....Brooklyn, N.Y.
Belzner, Theodore, New York City
Bensel, J. A......New York City
Berger, Bernt.....New York City
Bigelow, W. D....New York City
Binion, Joshua....New York City
Bissell, H....W. Medford, Mass.
Blakeslee, C...New Haven, Conn.
Boecklin, Werner, Jr. New York City
Boller, A. P......New York City
Bolton, R. P......New York City
Bonzano, A.....Philadelphia, Pa.
Bouton, G. Harold.Boonton, N. J.
Bowman, A. L....New York City
Boyd, James C....Boston, Mass.
Bradley, C. W.....Buffalo, N. Y.
Braine, L. F.....Brooklyn, N. Y.
Bramwell, G. W...New York City
Breuchaud, J.....Yonkers, N. Y.
Briggs, Josiah A..New York City
Brinckerhoff, H. W.New York City
Brown, T. E......New York City
Buck, L. L.......New York City
Buck, R. S........New York City
Bullock, Wm. D.Providence, R. I.
Burdett, F. A....Brooklyn, N.Y.
Bush, Edward W.Hartford, Conn.

Carll, David S..Washington, D. C.
Carney, Edward J.New York City
Carr, Albert......New York City
Cartwright, R...Rochester, N. Y.
Catt, George W...New York City
Cattell, William A., New York City
Chambers, R. H...New York City
Chase, F. L......Louisville, Ky.
Christian, G. L....New York City
Christy, George L.New York City
Clapp, L. R.....Hempstead, N.Y.
Clapp, Otis F....Providence, R. I.
Clark, George H...New York City
Clarke, T. C......New York City
Coffin, Amory.....New York City
Coffin, T. Amory..New York City
Cogswell, W. B....Syracuse, N.Y.
Cohen, Mendes... Baltimore, Md.
Colby, S. K.......New York City
Compton, A. G....New York City
Constable, H......New York City
Cooley, M. W.....Baltimore, Md.
Cooper, S. L......Yonkers, N. Y.
Cooper, Theodore..New York City
Corby, C. E ... .New York City
Cornell, George B.New York City
Corthell, A. B.....Boston, Mass.
Corthell, E. L.....New York City
Cotton, J. P.......Newport, R. I.
Craven, Alfred.Kingsbridge. N. Y.
Croes, J. James R.New York City
Crowell, Foster....New York City
Cuddeback, A. W..Paterson, N. J.
Cudworth, F. G..Brooklyn, N. Y.
Cummings, Noah..New York City
Curtis, F. S....New Haven, Conn.

Dalrymple, F. W. Hornellsville, N.Y.
Davis, A. L ..East Berlin, Conn.
Davis, Charles....Allegheny, Pa.
Davis, Robert B....Boston, Mass.
Dawley, E. P...Providence, R. I.
Dean, Luther....Taunton, Mass.

Deans, John S..Phœnixville, Pa.
Deyo, S. L. F.....New York City
Drake, A. B..New Bedford, Mass.
Dunham, H. F....New York City
Duryea, Edwin, Jr.
Brooklyn, N. Y.

Edwards, J. H. East Berlin, Conn.
Ellis, John W..Woonsocket, R. I.
Erlandsen, Oscar..New York City
Evans, M. E......New York City

Fanning, J. T. Minneapolis, Minn.
Farnum, H. H....New York City
Farrington, H.....New York City
Fisher, Clark......Trenton, N. J.
Fisher, Francis D. Cornwall, Ont.
Fisher, Wager...Bryn Mawr, Pa.
FitzGerald, D.....Boston, Mass.
Fort, E. J ......Brooklyn, N. Y.
Francis, George B..Boston, Mass.
Francis, H. N...Providence, R. I.
Frank, George W. New York City
Frazee, John H....New York City
French, A. H....Brookline, Mass.
French, J. B......New York City
Fritz, John......Bethlehem, Pa.
Frost, George H. New York City
Fuller, George W. New York City
Fuller, W. B......Malden, Mass.
Furber, Wm. C. Philadelphia, Pa.

Gartensteig, C....New York City
Gatchell, George S. Buffalo, N. Y.
Gay, Martin......New York City
Gibbs, George..Philadelphia, Pa.
Gifford, George E. New York City
Giles, Robert.....New York City
Goldmark, Henry..Detroit, Mich.
Gould, E. Sherman. Yonkers, N. Y.
Gowen, C. S......Sing Sing, N. Y.
Graham, C. H ...New York City.
Granbery, J. H..Elizabeth, N. J.
Grant, T. H.....Red Bank, N. J.
Graves, Edwin D., Hartford, Conn.
Gray, William....New York City
Green, B. R...Washington, D. C.
Greene, Carleton..New York City
Greene, G. S., Jr..New York City
Greene, J. N........Bangor, Me.
Gregory, C. E ....New York City
Gregory, J. H.....New York City
Greiner, J. E.....Baltimore, Md.
Grimm, C. R.......Elmira, N. Y.

Haight, Stephen S. New York City
Haines, H. S.....New York City
Hankinson, A. W., New York City
Hansel, Charles W. New York City
Harby, Isaac......New York City
Harris, C. M......New York City
Harrison, E. W. Jersey City, N. J.
Haskins, W. J. New Rochelle, N.Y.
Hawks, A. McL..Tacoma, Wash.
Hazen, Allen..... New York City
Hemming, D. W..New York City
Henry, Philip W..New York City
Hering, Rudolph..New York City
Herschel, Clemens, New York City
Hewitt, Charles E. Trenton, N. J.
Hill, Albert B., New Haven, Conn.
Hill, George......New York City
Hill, W. R........New York City
Himes, Albert J...Oswego, N. Y.
Hinds, F. A ...Watertown, N. Y.
Hoag, S. W., Jr...New York City
Hodgdon, F. W....Boston, Mass.
Hodge, Henry W. New York City
Honness, G. G....Paterson, N. J.
Hood, R. H ......New York City
Howe, E. W.......Boston, Mass.
Howe, Horace J...New York City
Hoxie, R. L...Washington, D. C.
Hoyt, John T. N..New York City
Hoyt, Wm. E....Rochester, N. Y.
Humphrey, R. L. Philadelphia, Pa.
Hunt, Chas. W....New York City
Hunt, Robert W....Chicago, Ill.
Hurtig, J. B......New York City
Hutton, W. R....New York City

Irving, Walter E..New York City
Ives, Arthur S...Brooklyn, N. Y.

Johnston, A. L....Richmond, Va.
Johnston, J. P...Brooklyn, N. Y.
Jonson, Ernst F...New York City
Judson, Wm. Pierson. Albany, N.Y.
Just, George A....New York City

Kahn, J..........New York City
Karner, W. J.......Chicago, Ill.
Kastl, Alex. E.....Clinton, Mass.
Katté, Walter
Ardsley-on-Hudson, N. Y.
Keith, H. C...New Haven, Conn.
Kelley, William D. New York City
Kelly, Olaf M.....New York City
Knap, J. M.......New York City
Kimball, George A. Boston, Mass.
Kuichling, E....Rochester, N. Y.

La Chicotte, H. A. New York City
Lant, Frank P....New York City
Leavitt, C. W., Jr., New York City
Lee, W. B....... Hillburn, N. Y.
Leffingwell, F. D..New York City
Leonard, H. R..Philadelphia, Pa.
Lesley, R. W...Philadelphia, Pa.
Leutzé, T. McC....Albany, N. Y.
Lewinson, M......New York City
Lewis, Nelson P. Brooklyn, N. Y.
Loomis, Horace...New York City
Low, George E....New York City
Lowinson, Oscar..New York City
Lucius, A.........New York City

McComb, D. E..Washington, D.C.
MacGregor, R. A..New York City
McKeever, Wm...New York City
McKenzie, T. H..Hartford, Conn.
McMinn, T. J.....New York City
Magor, H. Basil...New York City
Manley, Henry.... Boston, Mass.
Marburg, E....Philadelphia, Pa.
Marple, William M. Scranton, Pa.
Marstrand, O. J...New York City
Martin, C. C.....Brooklyn, N. Y.
Martin, K. L......New York City
Martin, Wisner B. New York City
Mead, Charles A...Newark, N. J.
Meem, J. C......Brooklyn, N. Y.
Melius, L. L.......Albany, N. Y.
Merryman, W. C. Brunswick, Me.
Meyer, Henry C...New York City
Miller, Hiram A...Clinton, Mass.
Miller, Rudolph P. New York City
Miller, Spencer...New York City
Mixer, C. A .Rumford Falls, Me.
Moisseiff, Leon S..New York City
Moore, Charles H. New York City
Moore, Wm. H. New Haven, Conn.
Mordecai, Aug...Cleveland, Ohio
Morse, C. M.......Buffalo, N. Y.
Moulton, Mace. Springfield, Mass.
Myers, C. H...... New York City
Myers, J. H., Jr..Brooklyn, N. Y.

Neumeyer, R. E..Bethlehem, Pa.
Nichols, O. F....Brooklyn, N. Y.
North, Edward P..New York City
Noska, G. A...... New York City

Oestreich, H. L., Jr.
New York City
Odell, F. S...Port Chester, N. Y.
Opdyke, S. B., Jr. Philadelphia, Pa.
O'Rourke, John F. New York City

Paine, George H.. New York City
Parker, A. McC...New York City
Parsons, George W. Steelton, Pa.
Parsons, W. B.... New York City
Pegram, George H. New York City
Peterson, P. A....Montreal, Que.
Peverley, Ralph...New York City
Pierce, William T..Boston, Mass.
Pitts, Thomas D..New York City
Plympton, G. W. Brooklyn, N. Y.
Polk, Wm. A......New York City
Porter, J. M......... Easton, Pa.

Potter, Alexander. New York City
Pratt, Mason D. .... Steelton, Pa.
Pratt, Wm. A. .. Philadelphia, Pa.
Prince, A. D. ..... New York City
Prout, H. G. ..... New York City
Pruyn, F. L. .... Brooklyn, N. Y.

Quincy, C. F. .... Chicago, Ill.

Reed, W. B. ...... New York City
Reynders, J. V. W. Harrisburg, Pa.
Richardson, T. F. . Clinton, Mass.
Ricketts, Palmer C. .. Troy, N. Y.
Ridgway, Robert. . New York City
Roberts, E. P. .... New York City
Roberts, P. ..... Philadelphia, Pa.
Roberts, William. Waltham, Mass.
Robinson, H. D. .. New York City
Rogge, J. C. L. ... New York City
Rood, H. M. .. Mt. Vernon, N. Y.
Rosenberg, F. .... New York City
Rowell, George F. New York City
Rowland, T. F. ... New York City
Rusling, G. M. Hackettstown, N. J.
Ryder, E. M. T. New Haven, Conn.

Schneider, A. ..... New York City
Schneider, C. C. .... Pencoyd, Pa.
Sherrerd, M. R. ... Newark, N. J.
Simpson, G. F. ... New York City
Skinner, Frank W., New York City
Sloan, R. I. .... New York City
Smith, E. F. ... Philadelphia, Pa.
Smith, J. Waldo. . Paterson, N. J.
Smith, Merritt H. . New York City
Smith, Oberlin. . Bridgeton, N. J.
Snow, J. P. .... Boston, Mass.
Spencer, W. T. ... Guilford, Conn.
Spilsbury, E. G. .. Trenton, N. J.
Staniford, C. W. . Brooklyn, N. Y.
Stearns, F. P. .... Boston, Mass.
Stevens, Alexander. New York City
Stern, E. W. ...... New York City
Stewart, John H. . New York City
Stowe, H. C. ...... New York City

Strachan, Joseph. Brooklyn, N. Y.
Swain, George F. .. Boston, Mass.

Tatnall, G. .... Wilmington, Del.
Taylor, Charles F. . Syracuse, N.Y.
Taylor, Lucien A. .. Boston, Mass.
Theban, J. G. .... New York City
Thompson, S. C. .. New York City
Thompson, Sanford E.
Newton Higlands, Mass.
Thomson, G. H. .. New York City
Thomson, John .. New York City
Thomson, T. K. ... New York City
Tillson, G. W. ... Brooklyn, N. Y.
Tingley, G. C. .. Providence, R. I.
Tinkham, S. E. .... Boston, Mass.
Tompkins, E. De V. New York City
Towne, J. M. . East Orange, N. J.
Travell, Warren B. . Orange, N. J.
Tribus, L. L ..... New York City
Trotter, Alfred W. . New York City
Trout, Charles E. New York City
Turner, D. L. .. Cambridge, Mass.
Turner, E. K. ...... Boston, Mass.
Tuska, G. R. ..... New York City

Ulrich, Daniel. ... New York City
Upham, Charles C. . New York City

Van Buskirk, C. R. New York City
Van Horne, J. G. . New York City
Vickers, T. McE. . Syracuse, N. Y.
Vielé, M. A. .... Wyncote, Pa.
von Leer, I. W. ... New York City
Vorce, C. B. ..... Hartford, Conn.

Wagner, B. M. ... Flatbush, N. Y.
Wagner, J. C. .. Philadelphia, Pa.
Waite, Guy B. .... New York City
Walker, Clement I. . New York City
Wallace, John F. .... Chicago, Ill.
Ward, C. D. .... New York City
Ware, R. W. ..... Plainfield, N. J.
Waterhouse, J. .... New York City
Watkins, F. W. White Plains, N. Y.

Webster, Albert L..New York City
Webster, G. S..Philadelphia, Pa.
Webster, Wm. R.Philadelphia, Pa.
Wegmann, E.....Katonah, N. Y.
Weiskopf, S. C....New York City
Wells, Joseph A...New York City
Wheeler, H. R....New York City
Whipple, G. C...Brooklyn, N. Y.
Whitney, F. O..... Boston, Mass.
Wiley. William H., New York City
Wilkes, J. K.New Rochelle, N. Y.
Williams, C. G...Brooklyn, N. Y.
Williamson, F. S..New York City
Wilson, C. W. S...New York City
Wisner, George Y..Detroit, Mich.
Wölfel, Paul L.....Pencoyd, Pa.
Wood, Henry B....Boston, Mass.
Woods, H. D. West Newton, Mass.
Worcester, J. R....Boston, Mass.
Wortendyke, N. D. Jersey City, N. J.

York, H. W.......New York City

MINUTES OF MEETINGS.

OF THE SOCIETY.

February 7th, 1900.—The meeting was called to order at 8.40 P. M.; Samuel Whinery, Director, in the chair; Charles Warren Hunt, Secretary, and present, also, 71 members and 10 visitors.

The minutes of the meeting of January 17th, 1900, as printed in *Proceedings* for January, 1900, were approved.

A paper by William B. Landreth, M. Am. Soc. C. E., entitled "The Improvement of a Portion of the Jordan Level of the Erie Canal," was presented by the author. A written discussion by George W. Rafter, M. Am. Soc. C. E., was read by the Secretary, and the subject was discussed orally by Messrs. Edward P. North, Allen Hazen, James Owen, George Hill, J. G. Tait, Samuel Whinery and the author.

Ballots were canvassed, and the following candidates declared elected:

As Members.

Edward James Beard, Mansfield, Ark.
Hiram Martin Chittenden, Sioux City, Ia.
Clarence Walter Hudson, Phœnixville, Pa.
Frank Henry Olmstead, Los Angeles, Cal.
Charles Jeremiah Parker, Watertown, N. Y.
John Henry Quinton, Los Angeles, Cal.

As Associate Members.

Thomas Johnstone Bourne, Tientsin, North China.
Thomas Bines Bryson, Philadelphia, Pa.
Charles Worthington Comstock, Denver, Colo.
Robert Duncan Coombs, Jr., Harrisburg, Pa.
Myron Edward Evans, New York City.
John Charles Lounsbury Fish, Palo Alto, Cal.
Rankin Johnson, Durango, Mexico.
Oscar Lowinson, New York City.
Ewald Schmitt, Washington, D. C.
James Lyall Stuart, St. Louis, Mo.
Henry Vier, New York City.
Elmer Zarbell, Chicago, Ill.

The Secretary announced the election of the following candidates by the Board of Direction on February 6th, 1900:

As Associate.

George Adam Weber, New York City.

As Juniors.

Marius Schoonmaker Darrow, Ithaca, N. Y.
Edmund Payton Ramsey, New York City.
George Reed Wadsworth, Albany, N. Y.

The Secretary announced the death of the following members: James David Moffet, elected Associate Member, November 4th, 1891; Member, February 7th, 1894; died November 3d, 1899. John MacLeod, elected Member, July 10th, 1872; died January 21st, 1900.

Adjourned.

February 21st, 1900.—The meeting was called to order at 8.40 P. M., Vice-President Rudolph Hering in the chair; Charles Warren Hunt, Secretary, and present, also, 97 members and 8 visitors.

A paper by Charles S. Gowen, M. Am. Soc. C. E., entitled, "The Foundations of the New Croton Dam," was presented by the author and illustrated with lantern slides.

The subject was discussed by E. Sherman Gould, M. Am. Soc. C. E., and a written discussion by George W. Rafter, M. Am. Soc. C. E., was presented by the Secretary.

The Secretary announced the death of the following members: William H. H. Benyaurd, elected Member November 3d, 1875; died February 7th, 1900. George H. Norman, elected Member February 17th, 1869; died February 4th, 1900.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

January 16th, 1900. (Adjourned meeting).—President FitzGerald in the chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Buchholz, Clarke, Haines, Manley, North, Thomson and Wisner.

The Secretary presented the report of the Committee to Recommend the Award of Prizes. *

It was resolved that the Norman Medal, the Thomas Fitch Rowland Prize, and the Collingwood Prize, be awarded in accordance with the recommendations of the Committee.

A report of the Committee appointed to report to the Board on the Acoustics of the Auditorium was received. †

The report was accepted, and the Secretary instructed to present it to the Society at the Annual Meeting.

Adjourned.

January 17th, 1900.—The Board met at 12.45 P. M., President Wallace in the chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Cartwright, Clarke, Deyo, FitzGerald, Hering, Knap, Manley, O'Rourke, Ricketts, Seaman, Turner and Wisner.

The following Standing Committees were appointed:

Finance Committee: Samuel Whinery, S. L. F. Deyo, Henry Manley, Robert Cartwright, Henry B. Seaman.

Publication Committee: Rudolph Hering, Palmer C. Ricketts, John F. O'Rourke, James D. Schuyler, Alfred Noble.

Library Committee: John A. Bensel, C. W. Buchholz, John Kennedy, Robert Moore, Chas. Warren Hunt.

A letter ballot was ordered for the election of a Secretary for the ensuing year.

Adjourned to meet on February 6th, 1900.

February 6th, 1900.—(Adjourned meeting.)—Vice-President Hering in the chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Bensel, Buchholz, Clarke, Deyo, Knap, O'Rourke, Ramsey, Seaman, Turner, Whinery.

Ballots in the matter of the election of a Secretary were canvassed with the following result: twenty-six ballots in all were received, all in favor of Chas. Warren Hunt for Secretary.

The Chair declared Chas. Warren Hunt elected Secretary of the Society.

Adjourned.

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* See page 27.
† See page 34

February 6th, 1900.—The Board met at 9 P. M., Vice-President Hering in the chair; Chas. Warren Hunt, Secretary, and present, also, Messrs. Bensel, Buchholz, Clarke, Deyo, Knap, O'Rourke, Ramsey, Seaman, Turner and Wisner.

Upon a report received from the Finance Committee the salaries of officers of the Society were fixed for the year and an appropriation made for the payment of other employees.

The Chairman of the Finance Committee presented informally a diagram showing statistics of the Society from 1886 to 1899, inclusive.

It was resolved that this diagram be published, for the information of members, in *Proceedings*.

The Committee of Arrangements for the Annual Convention reported progress.

Applications were considered and other routine business transacted.

One candidate for Associate and three for Junior were elected.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 A. M. to 10 P. M.

MEETINGS.

Wednesday, March 7th, 1900, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by Charles D. Marx, M. Am. Soc. C. E., Charles B. Wing, Assoc. M. Am. Soc. C. E., and Leander M. Hoskins, C. E., entitled "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company, at Ogden, Utah, Second Series," will be presented for discussion. This paper is printed in this number of *Proceedings.*

Wednesday, March 21st, 1900, at 8.30 P. M., a regular meeting will be held, at which a paper by George S. Webster and Samuel Tobias Wagner, Members, Am. Soc. C. E., entitled "History of the Pennsylvania Avenue Subway, Philadelphia, and Sewer Construction Connected Therewith," will be presented for discussion. This paper is printed in this number of *Proceedings.*

STATISTICS OF THE WORK OF THE SOCIETY, 1886 TO 1899.

The diagram on the opposite page was prepared for the purposes of the Finance Committee, and was brought to the attention of the Board of Direction, February 6th, 1900. The Board, in the belief that the information shown thereon would be of much interest to members, ordered that it should be printed in *Proceedings*.

The following explanatory notes are thought to be necessary to a full understanding of the diagram:

1. The curves of "Total Current Receipts" and "Total Current Expenses" do not include any unusual items. Legacies, donations, subscriptions or disbursements for special objects (such as the Engineering Congress in 1893), cost of or returns from the "Historical Sketch of the Society," receipts or payments on account of the new or old Society House, etc., etc., are excluded.

2. Owing to differences in methods of book-keeping, it has been impossible to arrive at figures which represent an absolutely correct comparison between years; nevertheless, the differences are so small as not to be in any way material.

3. For the years 1886 to 1892, inclusive, all of the salary of the Secretary was charged to "Salaries," or "Salary of Secretary," all other salaries being distributed to the various accounts, hence the curves showing the "Gross and Net Cost of Publications" for these years are somewhat lower than they should be.

4. Between 1886 and 1894 the publications were not always issued during the month or year of their date, and for the "Total Pages of Publications" and "Net Cost per Page" the figures used are the number of pages issued under date of each year.

5. It should also be noted that the total edition of publications increases practically in the same ratio as the total membership, the increase in the latter from 1886 to 1899 being about 125 per cent. This item is not taken into account in the curve of "Net Cost of Publications per Page."

6. The upward tendency of all the Expense Curves for 1898 and 1899 is due largely to the extra work of reclassifying and indexing the Library. It should also be mentioned that the Society House has during these years been kept open daily (except Sunday) for 13 hours instead of 8 as formerly, and that this materially increases the expense for attendance, as well as for the heating and lighting of the much more commodious quarters.

Some Statistics of the work of the

AMERICAN SOCIETY OF CIVIL ENGINEERS

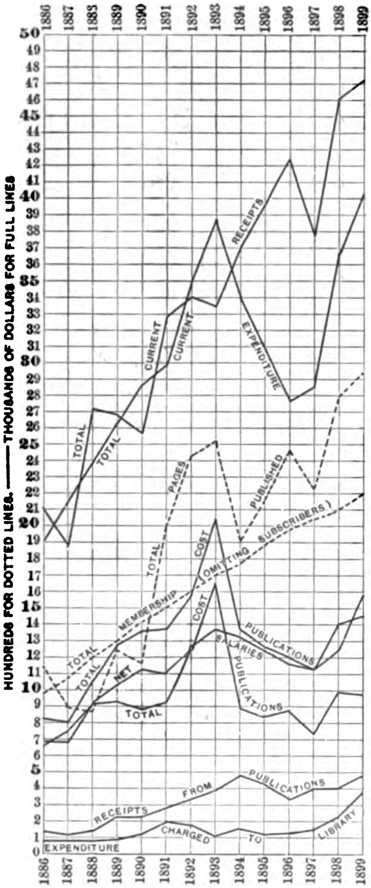
from 1886 to 1899.

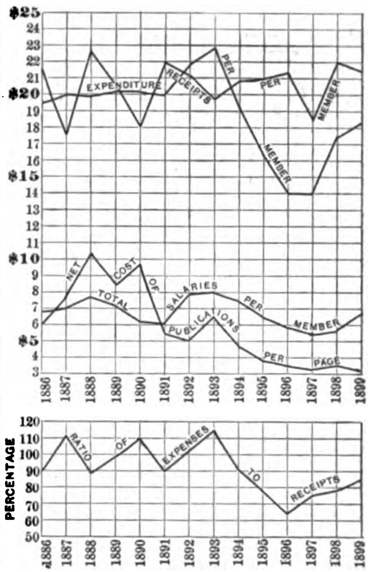
PREPARED UNDER DIRECTION OF THE FINANCE COMMITTEE

AND PRINTED BY ORDER OF THE

BOARD OF DIRECTION

FEBRUARY 1900





NOTES:

CURRENT RECEIPTS AND EXPENDITURES DO NOT INCLUDE EXTRAORDINARY ITEMS OF ANY KIND.

PAGES OF PUBLICATIONS ARE THOSE PUBLISHED UNDER THE DATE OF EACH YEAR.

1893 WAS THE YEAR OF THE INTERNATIONAL ENGINEERING CONGRESS AT CHICAGO.

NEW SOCIETY HOUSE PROJECTED 1895
" " " BUILDING BEGUN FALL OF 1896
" " " " FINISHED NOVEMBER 1897

NEW LIBRARY CLASSIFICATION AND INDEX STARTED 1898.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(January 9th, to February 12th, 1900.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c.
(14) *The Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*. Stockholm, Sweden.
(57) *Teknisk Ugeblad*, Christiania, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(60) *Bridges and Framed Structures*, 358 Dearborn St., Chicago, Ill., 30c.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

LIST OF ARTICLES.

Bridge.

Long Span Bridges. William H. Burr, M. Am. Soc. C. E. (15) Jan. 19.
Economies in Railway Bridge Design and Manufacture. J. Graham. (12) Jan. 26.
Some Light Highway Suspension Bridges. (14) Feb. 3.
Machinery in Bridge Erection. Charles Evan Fowler, M. Am. Soc. C. E. (10) Feb., 1900.
The Lehigh Valley Railroad Bridge at Easton, Pa. (14) Feb. 10.
The Zanesville, O., Timber Howe Truss Y-Bridge Built in 1881-2. (13) Jan. 25.
Design of a 175-Foot Counter-Balanced Plate Girder Swing Bridge. A. Reichmann. (4) Dec., 1899.
Construction of a 175-Foot Counterbalanced Plate Girder Swing Bridge. W. A. Rogers. (4) Dec., 1899.
A Contractor's Transfer Bridge. (14) Jan. 13.
New Railway Bridges in Paris. (12) Jan. 5.
Telescopic Drawbridge over the River Dee, at Queensferry, England. Thomas Walter Barber. (13) Jan. 18.
Erection of the Atbara River Bridge. (18) Jan. 13; (40) Jan. 12.
The New Colenso and Frere Bridges. (12) Jan. 19.
The Frere and Tugela Bridges. (11) Jan. 26.
On Hinged Concrete Bridges. (51) Serial, Jan. 6-10.
The Projected Bridge Across Lille Belt, Denmark. (55) Dec. 30.
Transport de Force par l'Electricité. Ponts Roulants Electriques, construits par la Société International d'Électricité à Liege (Belgique). (34) Jan., 1900.
Etude des Mouvements vibratoires dans les Ponts à Poutres Droites à une Travée et dans les Ponts Suspendus à Tablier Continus Simplement Appuyée aux Culées. M. Lebert. (43) Third trimestre.

Electrical.

Notes on Commutatorless Dynamo Design. H. E. Heath. (27) Feb. 10.
Electrical Supply and Alternator Design. Alton D. Adams. (27) Jan. 27.
Parallel Operation of Direct Coupled Alternators. W. L. R. Emmet. (27) Jan. 20.
The Factors which Determine the Design of Monophase and Polyphase Generators. B. A. Behrend. (27) Serial beginning Jan. 20, ending Feb. 3.
Double Current Generators. (27) Jan. 27.
Three Wire Distribution from One Machine. Edward Bretch. (27) Jan. 13.
The Hysteretic Qualities of Iron, Viewed from the Molecular Magnet Standpoint. Dr. Samuel Sheldon. (27) Feb. 10.
Compensation of Line Drop in Alternating Current Circuits. E. J. Berg. (27) Jan. 13.
A Graphical Treatment of the Effect of Magnetic Leakage on Transformer Regulation. F. G. Baum. (27) Jan. 13.
Stroboscopic Methods of Determining the Revolutions and Slip of Small Motors and the Frequency of Alternating Currents. (26) Jan. 5.
Notes on Maximum Demand Indicators. Louis J. Steele. (26) Jan. 5.
The Polyphase Induction Motor. Ralph D. Mershon. (27) Serial beginning Feb. 3, ending Feb. 10.
Alternating Current Power Motors. W. A. Layman. (47) Serial beginning Jan. 20, ending Feb. 3.
The Application of Niagara Power to the Work of the International Traction Company. (17) Feb. 3.
The Economics of Long-Distance Electric Power Transmission. Alton D. Adams. (13) Feb. 1.
Niagara Power Substations for Buffalo Electric Railway Service. (27) Jan. 13.
Alternate-Current Power Transmissions. C. Du Riche Preller. (11) Jan. 12.
Influence of Cheap Fuels on the Cost of Electrical Energy. R. E. Crompton. (11) Jan. 12.
Continuous and Multiphase Power Plants for Factory Use. (26) Jan. 26.
Electrical Equipment of the Berlin, Germany, Elevated Railway. (27) Feb. 3.
Electrical Equipment of United States Government Powder Factories. (11) Jan. 26.
The Future of Electric Illumination. Jean Wetmore. (24) Serial beginning Jan. 15, ending Feb. 12.
Relative Candle-Powers of Alternating and Direct-Current Enclosed Arc Lamps. (27) Jan. 13.
Leicester Corporation Electricity Works. (26) Jan. 26.
Plant of London Metropolitan Electric Supply Company. (27) Feb. 3.
Electrical Underground Construction. George B. Springer. (4) Dec., 1899.
Electrical Time Service. F. Hope Jones. (47) Serial beginning Jan. 13, ending Jan. 20.
Blasting by Electricity. Claude H. Smith. (28) Jan. 5.
Methods of Suppressing Arcs in Switches, Fuses, Etc. Ernest Kilburn Scott. (47) Serial beginning Dec. 16, ending Jan. 6.
Apparatus for Use in Experimental Fused Electrolytes. Alec. A. Beadle. (26) Serial beginning Jan. 19, ending Jan. 26.
Method of Testing the Resistance of Rail Bonds. (17) Feb. 3.
The "Bedford" Telephone Exchange, Brooklyn. (27) Jan. 20.
The "Central Energy" Multiple Switchboard, Independent Telephone Exchange, Parkersburg, W. Va. (27) Jan. 27.
The Development of Wireless Telegraphy. Patrick B. Delaney. (9) Feb., 1900.

Electrical—(Continued).

Electric Stations with Gas Engines as Prime Motors. M. Krone. (48) Jan. 13.
Electric Water-Power Station in Arboga. (56) Jan. 13.
Le Téléphone à Paris. A. Dennery. (33) Serial beginning Jan. 20, ending Jan. 27.

Marine.

The Modern Warship as Combining in Itself the Highest Results of Skill, Ingenuity and Scientific Knowledge. Rear Admiral George W. Melville, Hon. M. Am. Soc. C. E. (3) Feb., 1900.
Electrical Installations on Battleships. J. J. Woodward, U. S. N. (11) Jan. 5.
Coaling Vessels at Sea. (12) Jan. 19.
The Problem of Coaling Warships at Sea. Spencer Miller, M. Am. Soc. C. E. (9) Feb., 1900.
Steam Pipes Aboard Ships, Providing for Expansion. A. B. Willits. (10) Feb., 1900.
The Ore Carrying Fleet of 1900. Waldon Fawcett. (62) Feb. 1.
Ice Breakers in Polar Explorations. Edwin Swift Balch. (3) Feb., 1900.
The Strength of Spars and Rigging of Sailing Vessels. (12) Serial beginning Jan. 5, ending Jan. 19.
The Electrical Machinery of the New Sea Dock at Yminden. (53) Jan. 19.

Mechanical.

Circulation in Steam Boilers. (13) Jan. 20.
The Rating of Boilers for Heating Purposes. (14) Jan. 27.
Ripper's Mean Pressure Indicator. (11) Jan. 26; (12) Jan. 26.
Production and Utilization of Superheated Steam. R. S. Hale. (9) Feb., 1900.
The Watson Radial Water Tube Boiler. (20) Feb. 1.
Marine Mechanical Stokers. Christian Larsen. (62) Jan. 25.
Mechanical Stokers, SS. *Pennsylvania*. (12) Jan. 26.
Powdered Coal for Steam Boilers. (22) Jan. 26.
Measurement of Steam and Water (47) Serial beginning Jan. 27, ending Feb. 3.
Combustion and Forced Draft. R. B. Hodgson. (47) Serial beginning Dec. 30, ending Jan. 6.
Influence of Cheap Fuels on the Cost of Power. R. E. Crompton. (22) Feb. 2.
Mechanical Devices as Applied to Firing of Steam Boilers. W. E. Snyder. (58) Dec., 1899.
On the Determination of Volatile Combustible Matter in Coke and Anthracite Coal. Richard K. Meade and James C. Attix. (28) Serial beginning Jan. 12, ending Jan. 19.
Table Showing the Loss of Pressure in Steam Pipes. A. F. Nagle. (13) Jan. 25.
Friction of Steam Packings. Charles Henry Benjamin (11) Jan. 12.
Steam Consumption of an Automatic Engine at Exceptionally High Speeds. W. B. Rainsford and H. W. Crowell. (8) Jan., 1900.
Receiver Drop in Multiple Expansion Engines. Prof. R. L. Wrighton. (47) Jan. 27.
The Influence of the Indicator Diagram on the Design of Valve Gear. (12) Jan. 19.
Sulzer Triple-Expansion Engines for the Berlin Municipal Electric Lighting System. (13) Jan. 25.
Tests of Two 10 000 000-Gallon Pumping Engines. John A. Laird. (47) Jan. 13.
Central Condensing Plants for Iron Works. (62) Serial beginning Dec. 14, ending Jan. 11.
The "Abeille" Petroleum-Spirit Motor and Carburettor. (28) Jan. 5.
Graphical Method of Constructing the Entropy Temperature Diagram of a Gas or Oil Engine. Henry J. Eddy. (47) Jan. 13.
An Efficiency Test of a 125-Horse-Power Gas Engine. C. H. Robertson. (11) Jan. 26.
The Diesel Oil Engine. (11) Jan. 5; (62) Feb. 8.
Oil Engines and Motor Cabs. Anthony G. New. (12) Serial beginning Jan. 5, ending Jan. 12.
Blast Furnace Gas Engines. J. D. Lyon. (62) Jan. 18.
Pioneers in Using Blast Furnace Gas. (62) Jan. 25.
A Blowing Engine Operated by Blast Furnace Gases. (16) Feb. 10.
Six Hundred Horse-Power Blast Furnace Gas Motor and Blowing Engine. (11) Jan. 19.
A De Brouwer Coke Conveyor and a Carbureted Water Gas Plant at the Crystal Palace Gas Works. (24) Feb. 12.
The Manufacture of Electric Automobiles. (27) Jan. 13.
American Types of Automobiles. (27) Jan. 27.
European Types of Electric Automobiles. (27) Feb. 10.
English Electrical Vehicles. (28) Jan. 26.
The Simpson-Bodman Steam Lorry. (28) Jan. 19.
Some New Motor Vehicles. (28) Jan. 12.
The Bardon Motor Car. (28) Jan. 26.
Motor Wheels for Vehicles. (26) Jan. 26.
Condensation of Steam in Blower Systems of Heating. (14) Jan. 27; (13) Feb. 1.
A Novel Type of Bucket Pump for Mines, Wells, Boreholes, etc. (22) Feb. 2.
The Koster Air Compressor. (22) Jan. 19.
A Neat Design of Motor-Driven Air Compressor. (47) Jan. 27.
Electric Traveling Cranes. (11) Jan. 5.
One Hundred Ton Shear with Adjustable Boom. (13) Jan. 25.
150-Ton Electric Crane at the Imperial Dock in Bremerhaven, Germany. (13) Feb. 8.
Compound Road Locomotives for the War Office. (12) Jan. 26.
Some Types of Friction Clutches. (22) Jan. 12.

Mechanical—(Continued).

Making Carborundum. (62) Jan. 25
The Dellwik-Fleischer Water Gas Process. (11) Jan. 26.
The Removal of Dust from Boiler Rooms. Albert A. Cary. (13) Feb. 8; (14) Feb. 3.
Compression and Liquefaction of Gases. Arthur L. Rice. (47) Serial beginning Jan. 27, ending Feb. 3.
Screw Fans. Charles H. Innes. (11) Jan. 12.
The Calculation of Disk Fans. (14) Jan. 27.
Methods of Testing Blowing Fans. (14) Feb. 3; (13) Feb. 8.
On the Practical Use of Liquid Air. (48) Jan. 20.
The Second International Acetylin Exhibition in Budapest, 1899. (53) Serial, Jan. 12-26.
Uehling's Conveying Machinery for Foundries. Ed. A. Uehling. (50) Jan. 1.
The Calcium Carbide and Acetylin Congress in Nürnberg, Oct., 1899. B. Carlson. (56) Dec. 28.
Industrial Plants Operated by The Waterfalls at Terni, Italy. F. Wagner. (54) Dec.
La Pétrole en Europe. (36) Jan. 10.
Une Machine à Vapeur Géante a l' Exposition. Andre Mahoudeau. (36) Jan. 25.
Les Machines-outils. G. Richard. (37) Dec., 1899.
Automotrice à Vapeur (Système Valentin Purrey). H. Brosselin. (38) Jan., 1900.
Les Chaudières Aquatubulaires dans la Marine Americaine. M. Hachebert. (33) Jan. 13.

Military.

Guns of Position and Siege Guns for the War. (12) Jan. 5.

Mining.

Safety Pit Props. (12) Jan. 12.
Iron Ore from Mine to Furnace. Waldon Fawcett. (16) Jan. 20.
A New Method of Measuring Stopes. Fred T. Greene. (16) Jan. 27.
Sinking through Heavily Watered Strata. Richard Robinson. (22) Jan 19.
The Ocean Coal Company's Deep Navigation Pits at Trehanis. (22) Jan. 19.
Blasting with High Explosives. Harold Bonser. (22) Jan. 26.
The Origin and Progress of Gold Dredging in New Zealand. W. H. Cutten. (9) Feb., 1900.
The Union Copper Mines, Gold Hill, N. C. Dr. A. R. Ledoux. (16) Feb. 10.
The Schuyler Copper Mines, New Jersey. (16) Feb. 3.
Notes on the Aurex Sluice. (16) Feb. 3.
Les Nouvelles Installations de la Compagnie de Witkowitz aux Mines de Dombrau (Autriche). H. Schmerber. (33) Jan. 20.

Municipal.

Failures in Asphalt Pavements. (24) Jan. 22.
Brick Paving in Iowa. (14) Feb. 3.
Municipal Tunnel Driving by Day Labor, Chicago. Paul G. Brown. (14) Feb. 3.

Railroad.

Momentum Grades. C. Frank Allen, M. Am. Soc. C. E. (15) Jan. 12.
The Early Railways of Surrey. (12) Serial beginning Jan. 5, ending Jan. 19.
Track Elevation in Chicago. (13) Jan. 11.
Locating the Center of Gravity of a Locomotive. (13) Jan. 18.
Canadian Pacific Locomotive Counterbalancing. (40) Feb. 2.
Cast Steel Driving Wheels. (25) Feb., 1900.
Locomotive Tenders. William Forsyth. (25) Feb., 1900.
Improvements in Locomotive Driver Brakes. (25) Feb., 1900.
Some Recent Electric Freight Locomotives. Burcham Harding. (13) Jan. 25.
Siege Train for the War. (12) Jan. 19.
What Does it Cost to Run Trains at High Speed? F. A. Delano. (13) Jan. 25; (18) Jan, 19.
The Laffas Patent Apparatus for Stopping Trains. (21) Feb., 1900.
Automatic Block Signal System in Lowell. (17) Feb. 3.
Automatic Block Signaling. Edward C. Carter. (15) Jan. 12.
Ventilating Passenger Cars on the Pennsylvania. (15) Feb. 9.
Building Large Embankments in the Boone County Railway (C. & N. W. Ry. System). (13) Feb. 8.
The Peoria and Pekin Terminal Railway. (13) Feb. 8.
The Extension and Alteration of the Belfast Terminus of the Belfast and Northern Counties Railway. (21) Feb., 1900.
Recent Extensions of the Bergen County Traction Company. (17) Feb. 3.
New Equipment for the Rapid Railway of Detroit. (17) Feb., 1900.
A Combination Steam and Electric Railway in Illinois. (17) Feb. 3.
The Metropolitan Railway in Paris. (12) Jan. 26.
Some Causes of Excessive Heating in Bearing Metals. Robert Job. (25) Feb., 1900.
Train Resistance due to Rail Sanding. (12) Jan. 26.
Economic Railways for Country Districts. E. E. Russell Tratman, Assoc. M. Am. Soc. C. E. (13) Feb. 1.
British Tramway Development. J. Clifton Robinson. (10) Feb., 1900.
Facing Points and Slip Switches. A. H. Rudd. (13) Jan. 11; (18) Jan. 13; (40) Jan. 12.
Rail for General Road Traffic. (55) Jan. 6.
The Aerial Overhead-Track Railroad between Barmen and Elberfeld. (51) Dec. 23.

Railroad—(Continued).
My Journey on the Siberian Railroad. N. Post. (53) Serial, Jan. 5–12.
Les Transports Electriques de l'Exposition de 1900. Emile Dieudonné. (36) Jan. 25.
Le Metropolitan de Berlin (Berliner Stadt-und-Ringbahn). Paul Haag. (38) Jan., 1900.
Les Transports en Commun dans Paris et sa Banlieue. Charles Jean. (33) Jan. 18.

Sanitary.
Erection of a Drainage Plant. (12) Jan. 5.
The Purification of Waste Water from Factories. (11) Jan. 12.
The Sewerage of Edinburgh, Scotland. James H. Fuertes, M. Am. Soc. C. E. (14) Jan. 13.
Sewage Disposal at Alliance, Ohio. (14) Jan. 13.
The Scott-Moncrieff System of Sewage Disposal. (12) Jan. 26.
Sewage Filtration Through Coal. (14) Feb. 3.
New Sewage Farm for the St. Denis Ward at Montreal. (13) Jan. 25.
The Bacterial Treatment of Sewage at Sutton, England. (14) Jan. 27.
Bacterial Sewage Disposal at East Cleveland. (14) Feb. 10.
Sewage Disposal at Chichester, England. James H. Fuertes, M. Am. Soc. C. E. (14) Feb. 10.
Garbage Collection and Disposal at Moline, Ill. Edward Kittilsen. (13) Feb. 8.
The Utilization of Garbage for Fuel. (14) Jan. 20.
Refuse and Its Calorific Power. Ed. C. de Segundo. (26) Jan. 12.
The Modified Horsfall Crematory. (14) Feb. 3.
The Barren Island Garbage Reduction Works, Greater New York. (13) Feb. 1.
The Public Baths of Bilston, Staffordshire, England. James H. Fuertes, M. Am. Soc. C. E. (11) Jan. 20.

Structural.
A New Extensometer. William A. Kenerson. (13) Jan. 25.
Review of Experimental Data on Impact Tests of Material in Tension. (13) Feb. 1.
Supports for Beams in Tests of Transverse Strength. William T. Magruder. (13) Jan. 11; (8) Feb., 1900.
The Softening of Portland Cement Plaster. (14) Feb. 3.
Tests of a Reinforced Concrete Floor. (14) Feb. 3.
Concrete Dock Construction at the South Works of the Illinois Steel Company. Victor Windett (4) Dec., 1899.
Tests of the Constancy and Volume of Portland Cement, (14) Serial beginning Jan. 20, ending Jan. 27.
The Talbot Continuous Open Hearth Steel Process. (20) Feb. 8.
The Japanese Imperial Steel Works. Emile Schrödter. (22) Feb. 2.
The Manufacture of Structural Steel in the United States. F. H. Kindl. (10) Feb., 1900.
The Color Names for High Temperatures. Henry M. Howe. (16) Jan. 20.
Iron Manufacture in the United States. John Fritz, Hon. M. Am. Soc. C. E. (11) Serial beginning Jan. 5, ending Jan 12.
The Design of Columns for Sheds Exposed to Wind, and of Concrete Base for Columns. (13) Jan. 1.
The Mormon Tabernacle, Salt Lake City. W. P. Hardesty. (14) Jan. 27.
The Reconstruction of the Manhattan Bank Building. (14) Feb. 3.
The Design of a Modern Apartment House. (14) Jan. 20.
A Large Steel Grand Stand. (14) Feb. 10.
Street Front Reconstruction. (14) Feb. 10.
The Design and Construction of a Modern Central Lighting Station. H. H. Humphrey. (47) Serial beginning Dec. 30, ending Jan. 6; (13) Jan. 18.
The New Works of the Harrisburg Foundry and Machine Works. (14) Jan. 27.
The New United States Navy Yard Coaling Station at New London, Conn. (13) Jan. 18.
The Power House of the St. Lawrence Power Company, Massena. (14) Feb. 10.
The Kalamazoo Valley Electric Company's Plant. (14) Jan. 13.
Municipal Lighting and Sewerage Plant, Concord, Mass. (27) Jan. 13.
The New Street Railroad Power Stations in New York. (15) Jan. 12.
The Niagara Falls Hydraulic Power and Manufacturing Company's Plant. (14) Jan. 20.
Fire Protection. Francis B. Stevens. (8) Jan., 1900.
A Belt Conveyor for Foundation Work. (14) Jan. 13.
Underpinning Shop Walls. (14) Jan. 13.
The Durability of Paint. (24) Jan. 29.
Foundations for Metal Tanks. R. J. Milbourne. (24) Jan. 22.
A Builder's Electric Traveling Derrick. (14) Feb. 10.
The Reconstruction of a Dry Dock. (14) Jan. 20.
Hot Air Furnaces for Small Schoolhouses. (14) Jan. 20.
A Hot Air Furnace Heating System. William O. Ludlow. (14) Feb. 10.
Ventilation and Heating of the Rookwood Pottery, Cincinnati. (14) Jan. 13.
Plumbing in the Dun Building, New York. (14) Jan. 20.
Maçonneries avec Joints Métalliques Coulés. M. Tavernier. (43) Third trimestre, 1899.
Logements à Bon Marché de la Société Anonyme Rémoise. (35) Jan., 1900.
Notice sur les Travaux de construction du troisième Bassin de Radeub de Mississy. M. Guiffart. (43) Third trimestre, 1899.

Topographical.
Lister's Inclinometer Theodolite. (11) Jan. 12.
The Rectangular System of United States Public Land Surveying. Charles L. Du Bois. (7) Dec., 1899.

Water Supply.

Experiments on the Flow of Water Over Bell-Mouthed Pipes. John Barr. (47) Jan. 6.
Ozone as a Water Purifier. (14) Feb. 8.
The Relative Values of Ozone and Slow Sand Filtration as a Means of Purifying Water. (13) Feb. 8.
The Water Purification Plant at Albany, N. Y. (13). Jan. 11.
Silica Standards for Determining Turbidity. (14) Jan. 27.
Silica Standards for the Determination of Turbidity in Water. George C. Whipple, Daniel D. Jackson. (7) Dec., 1899.
On the Necessity of Cultivating Water Bacteria in an Atmosphere Saturated with Moisture. George C. Whipple. (7) Dec., 1899.
A Russian Report on Mechanical Filters. (14) Jan. 20.
The Melbourne Water Supply. (11) Jan. 5.
The Water-Works of Portland, England. (14) Jan. 13.
Florence, Colo., Water-Works. R. P. Garrett. (14) Feb. 10.
The Use of Wells in Washington, D. C. (14) Jan. 13.
Water Meters of the Present Day; with Special Reference to Small Flows and Waste in Dribbles. William Schönheyder. (12) Feb. 2.
Standpipe Failure at Collingswood, N. J. (14) Jan. 20.
Covered Reservoirs. W. S. Shields. (13) Feb. 1.
The Wachusett Reservoir. (14) Jan. 20.
Outlet Valves, Burrator Reservoir, Plymouth, England. (14) Feb. 10.
A Large Crib Dam, Butte, Mont. (14) Feb. 3.
Exploration for Bedrock at Gila River Dam Sites with Diamond Core Drills. J. B. Lippincott. (13) Jan. 18.
Irrigation in Jeypore State. (14) Jan. 13.
The Second Water-Works of Vienna. Fr. Borkowitz. (53) Jan. 26.
Examples of Pumping Engines. H. v. Bavier. (48) Jan. 6.
The Water-Works of the City of Prenzlau. H Scheven. (48) Jan. 13.

Waterways.

The Lift Lock near Henrichenburg, Germany. (14) Jan. 13.
Shore Improvements at Blackpool. (11) Jan. 19.
Discharge Measurement of the Niagara River at Buffalo, N. Y. Clinton B. Stewart. (4) Dec., 1899.
Combined Gas Light and Bell Buoy. Robert M. Dixon. (8) Jan., 1900.
A New Type of Excavating Machine. S. H. Lea, M. Am. Soc. C. E. (13) Feb. 8.
Concerning the Actual Condition of the Panama Canal. Charles Paine, M. Am. Soc. C. E. (9) Feb., 1900.
The Development of German Canals in Latter Years. Fr. Enblom. (56) Jan. 20.
Navigation Interieure. La Traction Electrique sur les Canaux. Raoul Dubreuil. (36) Jan. 10.
Étude du Régime de la Marée dans la Manche. Léon Bourdelles. (43) Third trimestre 1899.
Transformation du Canal Lateral à la Loire E. Rouyer. (33) Jan. 13.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

HANDBOOK OF TESTING MATERIALS. FOR THE CONSTRUCTOR.

Part I, Methods, Machines and Auxiliary Apparatus. Vol. I, Text; Vol. II, Illustrations. By Professor Adolf Martens, Director of the Royal Testing Laboratories at Berlin and at Charlottenburg. Authorized Translation and Additions. By Gus C. Henning. Cloth, 9 x 6 ins., 2 vols., illustrated. New York, 1899. John Wiley & Sons. $7.50.

The author states in the Preface that his book "is designed to be a counsellor to the constructor in all questions relating to the properties of his materials of construction." His work is divided into two volumes. This one relates to the general properties of materials of construction, and especially to the art and science of testing materials as applied to machinery and superstructure. There is added a presentation and discussion of the most important types of testing machines and auxiliary apparatus. There is an index of thirteen pages.

THE COST OF LIVING

As Modified by Sanitary Science. By Ellen H. Richards. Cloth, 8 x 5 ins., 121 pp. New York, John Wiley & Sons, 1899. $1.00.

The headings of chapters are: Standards of Living; The Service of Sanitary Science in Increasing Productive Life; Household Expenditure; Rent or Value and Furnishings; Operating Expenses; Fuel, Light, Wages; Food; Clothing in Relation to Health; The Emotional and Intellectual Life; The Organization of the Household.

WATER-SUPPLY ENGINEERING.

The Designing, Construction and Maintenance of Water-Supply Systems, Both City and Irrigation. By A. Prescott Folwell, M. Am. Soc. C. E. Cloth, 9 x 6 ins., 562 pp., illus. New York, John Wiley & Sons, 1900. $4.00.

The ground covered by this volume is indicated by the contents, as follows: Part I. Designing: Requisites of a Supply, Quality; Quantity; Sources of Supply; Rainfall; Surface-Water; Rivers and Lakes; Ground-Water; Gravity Systems, Pumping Systems; Hydraulics; Dams and Embankments; Purification of Water; Pumping and Pumping Engines; Designing. Part II. Construction: Supervision and Measurement of Work; Practical Construction. Part III. Maintenance: Reservoirs, Head-Works and Intakes; Pumping Plants and Filters; Pipes and Conduits; Clerical and Commercial. There is an index of fifteen pages.

THE MANUAL OF AMERICAN WATER-WORKS, 1897.

Compiled from Special Returns; Containing the History and Description of the Source and Mode of Supply, Pumps, Reservoirs, Stand-Pipes, Distribution Systems, Pressures, Consumption, Revenue and Expenses, Cost, Debt and Sinking Fund, etc., of the Water-Works of the United States and Canada; With Summaries for each State and Group of States. Edited by M. N. Baker, Associate Editor of *Engineering News*. Cloth, 9 x 6 ins., 611 pp. The Engineering News Publishing Co. New York, 1897. $3.00.

This is the fourth issue of this work and contains descriptions of all water-works completed, under construction, or projected in the United States and Canada, up to the close of 1896. This book contains a tabulated statement of the water rates charged in over 1 250 cities and towns.

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ADDITIONS.

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Bacon, James Hayward, U. S. Asst. Engineer, St. Augustine, Fla.		Jan. 3, 1900
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Chittenden, Hiram Martin, Secretary and Disbursing Officer, Missouri River Commission, Sioux City, Iowa.		Feb. 7, 1900
Glasgow, Arthur Graham, 9 Victoria St, London, S. W., England.		Jan. 3, 1900
Hegeman, William Wallace Cons. Eng., 228 Bay St., East, Savannah, Ga.		Feb. 6, 1884
Hudson, Clarence Walter, Asst. Eng., Phœnix Bridge Co., Phœnixville, Pa.	Assoc. M. M.	Nov. 7, 1894 Feb. 7, 1900
Parker, Charles Jeremiah, Div. Eng., R. W. & O. Div., N. Y. C. & H. R. R. R., Watertown, N. Y.		Feb. 7, 1900
Touceda, Enrique Augusto, 51 State Street, Albany, N. Y.		Jan. 3, 1900

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Coombs, Robert Duncan, Jr., Box 290, Harrisburg, Pa.	Jun. Assoc. M.	May 2, 1899 Feb. 7, 1900
Ives, Arthur Stanley, 33 Sidney Place, Brooklyn, N. Y.	Jun. Assoc. M.	Jan. 7, 1896 Jan. 3, 1900
Lowinson, Oscar, 39 Cortlandt St., New York City	Jun. Assoc. M.	May 3, 1892 Feb. 7, 1900
Oestreich, Henry Lewis, Jr., 2092 Bathgate Ave., Tremont, New York City.	Jun. Assoc. M.	Oct. 4, 1892 Dec. 6, 1899
Vier, Henry, Asst. Eng., N. Y. C. & H. R. R. R., Grand Central Station, New York City.		Feb. 7, 1900
Zarbell, Elmer 4132 Ellis Ave., Chicago, Ill.	Jun. Assoc. M.	Jan. 3, 1899 Feb. 7, 1900

JUNIOR.

Ramsey, Edmund Payton,
174 West 126th St., New York City.................... Feb. 6, 1900

Wright, Moses Hannibal,
Asst. Eng., Louisville & Nashville Terminal Co., Nashville, Tenn.................. .... Jan. 2, 1900

DEATHS.

Benyaurd, William H. H...Elected Member Nov. 3d, 1875; died Feb. 7th, 1900.

MacLeod, John.............Elected Member July 10th, 1872; died Jan. 21st, 1900.

Norman, George H.........Elected Member Feb. 17th, 1869; died Feb. 4th, 1900.

Vol. XXVI. FEBRUARY, 1900. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXVI. FEBRUARY, 1900. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH. SECOND SERIES.

By CHARLES D. MARX, M. Am. Soc. C. E., CHARLES B. WING, Assoc. M. Am. Soc. C. E., and LEANDER M. HOSKINS, C. E.

TO BE PRESENTED MARCH 7TH, 1900.

I.—OBJECT AND METHODS.

General Plan.

The following experiments made in June and July, 1899, are supplementary to those made by the authors in August, 1897, and described in a previous paper.* The main object was the same as before; to determine the relation between the mean velocity of flow in the pipe and the loss of head between certain definite points. The methods used, being in general the same as in the former work, need not be described fully here; it will suffice to indicate the points of difference in the apparatus used, and in the methods of making and reducing the observations.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited, from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

* *Transactions*, Am. Soc. C. E., Vol. xl, p. 471.

The experiments of 1897 upon the steel pipe utilized a length of about 4 400 ft.* In the wood pipe experiments a length of 2 710 ft. was used. The long section of wood pipe above Tunnel No. 7† was not used because the overflow at that point prevented securing static conditions in the pipe above.‡ When the present series of observations was made, it was found possible to stop the overflow by adjusting flash boards properly at the relief shaft and at the dam. Measurements of the loss of head in a length of about 22 700 ft. of the wood pipe above Tunnel No. 7 were therefore made. In addition, a limited number of experiments was made upon the portion of wood pipe used in 1897, as well as a new series upon the steel pipe. A velocity of flow, materially higher than in the preceding series, was secured.

Pressure Measurements.

Six pressure stations, located at the ends of the three portions of pipe above mentioned, were occupied. For convenience of reference, these stations have been designated by the numbers 1, 2, 3, 4, 5 and 6, beginning at the lower end of the steel pipe. Mercury gauges were used at all stations except No. 6, at the upper end of the long section of wood pipe. At this point the pressure was so small that a water piezometer was used.

At Stations Nos. 4 and 5 were placed the gauges used in 1897, and their description need not be repeated here. The only change made in these gauges was the attachment of fixed scales for the purpose of reading the position of the mercury in the reservoirs. The gauges used at Stations Nos. 1, 2 and 3 were open manometers similar to the others, but of a modified design.

In the former experiments, it was found difficult to make the gauges absolutely mercury-tight under the high pressure existing at the power-house, and this difficulty was carefully guarded against in planning the new series. The new gauges were made stronger than the old, and were tested at the highest pressure under which they were to be used. Means were also provided for determining fluctuations in the level

* The length was 4 427 ft. during a part of the series, and 4 367 ft. during the remainder.

† For plan and profile, showing the position of this tunnel, see paper by Henry Goldmark, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 246.

‡ Static readings of the gauges could, of course, be dispensed with if the difference of level between the gauges were known with sufficient exactness. To determine this difference with the requisite accuracy by running a line of levels would have involved such an amount of labor and time as to put this method out of the question.

of the mercury surface in the manometer reservoir, so that, even if leakage of mercury occurred, the observation of the height of the mercury column would not be vitiated. For this purpose the gauge was provided with a glass tube placed vertically at the side of the reservoir and communicating with it at top and bottom (Fig. 1), and by the side of this tube was placed a fixed scale. The position of the

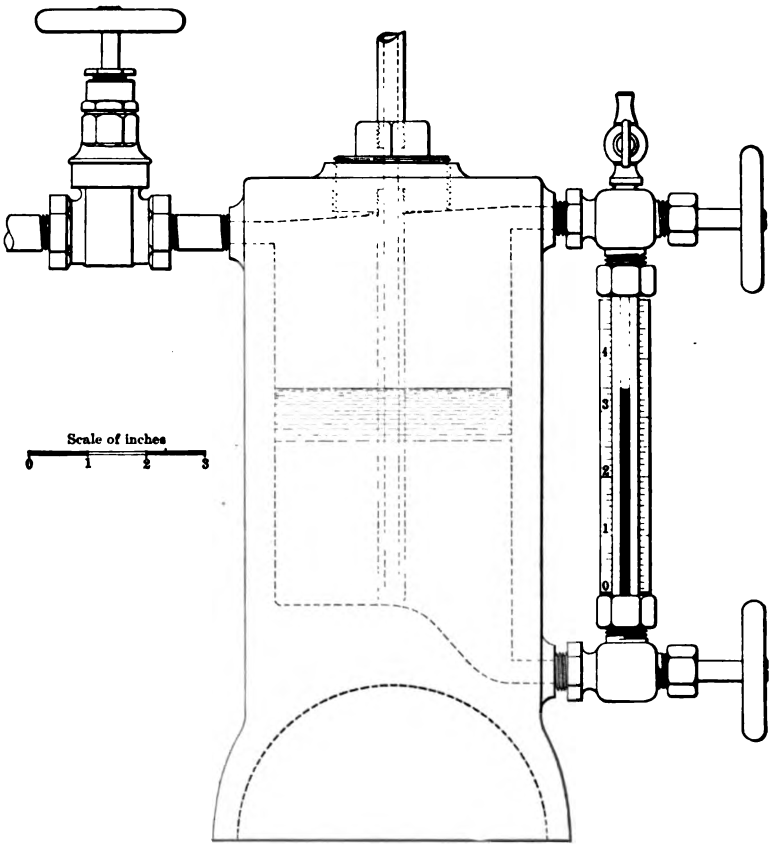

FIG. 1.

top of the mercury in this tube could be read with the same degree of precision as that of the top of the mercury column in the open tube The scales used were in all cases graduated to hundredths of a foot, and the third decimal place was estimated in taking readings.

As in the previous experiments, the long mercury column at Station No. 1 was provided with a water jacket carrying running water, to insure

a uniform temperature. At other stations the temperature was determined by thermometers placed beside the mercury columns.

The specific gravity of the mercury used, in terms of water from the pipe, was determined for the authors in the chemical laboratory of the Ogden Sugar Company, through the courtesy of the Superintendent, Mr. H. T. Dyer. The value adopted was 13.57.*

The pipe leading from the pressure section was in every case given a continuous upward inclination to the point of communication with the manometer reservoir, and was provided with a blow-off cock at the highest point, for the purpose of keeping this connecting pipe free from air. In the case of Manometer No. 1, this pipe was necessarily of considerable length, but the water in it could be completely changed by opening the blow-off for a few seconds. The vitiation of the results by air would require a sufficient accumulation to completely fill the cross-section of the pipe. It is reasonably certain that no such accumulation occurred in any piezometer. There was, in fact, no indication at any time that any important amount of air was carried by the water.

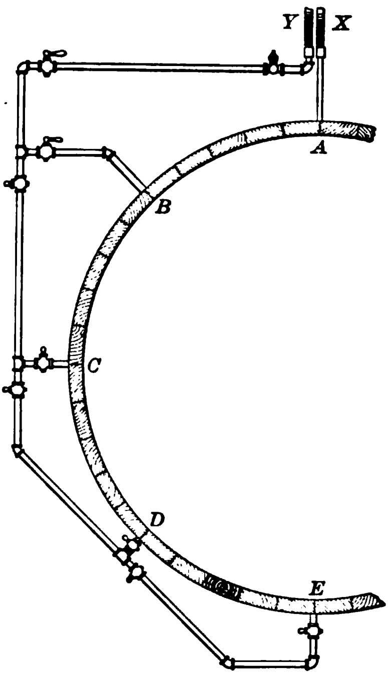

FIG. 2.

The attachment of piezometers to the main pipe was made in the same way as in the former series of observations,† except in the case of the water piezometer at Station No. 6. At this section the pipe was tapped at five points, *A*, *B*, *C*, *D*, *E* (Fig. 2). Two vertical glass tubes

* This value is believed to be reliable to within one-twentieth of 1 per cent. In reducing the former series of experiments, a value of 13.6 was used, the authors believing themselves to be justified in assuming, without an experimental determination, that this value was correct to the degree of accuracy demanded by the nature of the experiments. The difference between 13.57 and 13.6 would mean a difference of less than one-fourth of 1% in the values found for the loss of head. The authors have yet to learn of any experiments on flow in large pipes which can claim such a degree of accuracy as this figure represents, and do not claim such accuracy for their own experiments.

† *Transactions*, Am. Soc. C. E., Vol. xl, p. 475.

X and *Y* were used, the former communicating with the pipe at the single point *A*, the latter arranged to communicate with any one or more of the tubes running to *B*, *C*, *D*, *E*. The object of this arrangement was to test whether the indication of the piezometer was affected by the position of the point of attachment. For this purpose, simultaneous readings were taken of the water columns *X* and *Y*, the latter being in communication with the pipe at any one, or any combination, of the points *B*, *C*, *D*, *E*. The results showed a small difference between the reading of *X* and *Y*, the former being in all cases a little the higher. The oscillations of the two columns (which were somewhat rapid and not simultaneous) prevented a precise determination of the amount of this difference and of its variation with the velocity of flow. The best observations showed a difference of 0.02 ft. with a velocity of 4.7 ft. per second. No change in the reading *Y* appeared to be produced by changing the combination of points of communication with the pipe. It would seem, therefore, that the difference in the readings of *X* and *Y* was due to some accidental circumference affecting the connection at *A*. The reading *Y* was used in all cases in the observations for determining loss of head in the pipe.

Measurement of Rate of Discharge.

The rate of discharge was determined as before, by attaching difference-gauges to the two Venturi meters. The two difference-gauges were placed side by side, so that both could be read by a single observer (Fig. 6, Plate XV.). The connecting pipes were laid on a continuous up-grade from the pressure sections to the gauges, blow-off valves being placed at the summits to insure freedom from air.

Loss of Head in Meters.

The difference-gauges were triple, showing pressure-differences for three sections: upper section of Venturi, throat of Venturi, and full-sized section below Venturi. The pressure-difference for the first and third sections thus showed the loss of head between those points.

Programme of Tests.

In carrying out the observations, it was necessary to secure simultaneous observations of the rate of discharge and the pressure at each of the points between which the loss of head was to be computed. For this purpose, observers stationed at the several gauges in use took

PLATE XV.
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MARX, WING AND HOSKINS ON FLOW OF WATER.

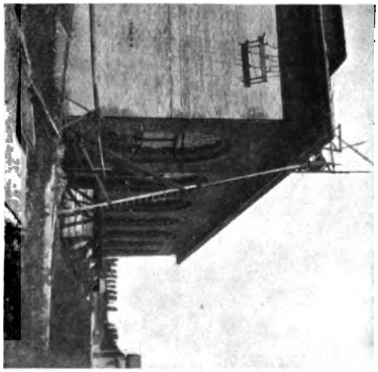


FIG. 1. MANOMETER No. 1. (1899.)

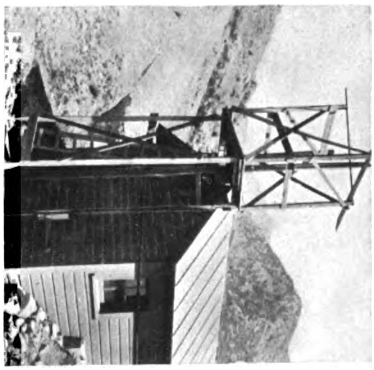


FIG. 2. MANOMETER No 2. (1899.)

FIG. 3. MANOMETER No. 3. (1899.)

FIG. 4. MANOMETER No. 5. (1899.)

FIG. 5. MANOMETER No. 6. (1899.)

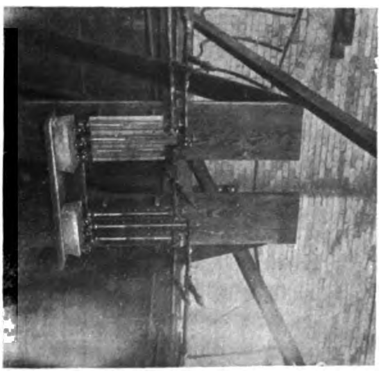


FIG. 6. DIFFERENCE GAUGE. (1899.)

readings at short intervals (generally one minute or less), throughout a period previously agreed on. During each period it was aimed to maintain a uniform rate of discharge. Because of fluctuations in the consumption of power by customers, the amount of water used by the wheels could not always be kept as constant as was desired, and it was necessary in most cases to use the average of readings, which varied somewhat during the interval in question. An interval of 20 minutes was adopted as the standard period covered by an "observation," and in most cases a nearly uniform flow was maintained during four consecutive 20-minute periods. The readings obtained during each 20-minute period were averaged, giving one "observation;" and, when the variation in the flow was small during several consecutive observations, these were combined to form a "group." The amount of difference between the observations of any group may be seen by reference to Table No. 1, to be explained below. At the beginning of each 20-minute interval the blow-off valves were opened for a sufficient time to insure freedom from air in the gauges and in the connecting pipes.

Reduction of Observations.

The method of reducing the observations was somewhat different from that used before;* the difference, is however, in form rather than in substance.

(1.) *Reduction of Manometer Reading to Equivalent Water Column.*—Each manometer observation was reduced to an equivalent water-piezometer reading. For this purpose a zero or datum level is assumed for each gauge, the position of which is arbitrary, but which it is convenient to take as the zero of the lower fixed scale.

The data furnished by a manometer observation are the following: Height of top of mercury column above zero of upper scale; height of mercury surface in reservoir above zero of lower scale; temperature of mercury column. Each of these quantities is found by averaging the readings taken throughout an observation period. When the pressure was very unsteady, the readings were plotted before averaging; a comparison of the plotted results obtained from the different gauges being of assistance in the selection of the exact readings to be used. In most cases the readings were sufficiently steady to make plotting unnecessary.

* *Transactions*, Am. Soc. C. E., vol. xl., p. 480.

The vertical distance between zeros of the two scales is a known constant for each gauge. Adding to this the upper reading and subtracting the lower reading, the result is the actual vertical length of the mercury column. This must be reduced to an equivalent column at a standard temperature. Strictly, this temperature should be that of the water in the portion of pipe under experiment; practically, it makes little difference what temperature is selected as the standard, except that variations in the water temperature must be taken into account, if of sufficient amount to affect the results materially. When the experiments were begun, the temperature of the water was in the neighborhood of 10° Cent., and this was chosen as the standard. The reduction to this standard temperature is made by applying the factor $1 - 0.00018\,(T - 10)$, T being the temperature of the mercury column in degrees Cent., and 0.00018 being a sufficiently exact value of the coefficient of expansion of mercury within the range of temperature found in the experiments.

The mercury column is reduced to water by multiplying by the specific gravity of mercury, which, as previously stated, was found to be 13.57. To the height of water, thus computed, must be added the lower scale reading, the zero of this scale being taken as the fixed piezometer datum.

Changes of temperature of the water in the pipe must be considered next. The difference in level of the two pressure stations on the steel pipe is about 300 ft. The difference between the pressures at these two stations will be changed appreciably by a change of even a few degrees in the temperature of the water. Thus, the coefficient of expansion of water in the neighborhood of 10° Cent. is very nearly 0.000084. A change of 4° in the temperature (say from 8° to 12°) would therefore cause the relative level of piezometers at the two points to change by about $300 \times 4 \times 0.000084 = 0.1$ ft. (very nearly). Observations of the temperature in the waste flume below the power-house showed, during the period covered by the experiments, a variation between 9.5° and 14° Cent. It is apparent, therefore, that this cause probably had an appreciable influence upon the gauge readings at Stations Nos. 1 and 2.

From observations of the temperature in the flume, an estimate was made of the mean temperature of the water in the steel pipe during each observation, and the corresponding correction was applied

TABLE No. 1.

Piezometer 4-5 Ft.	Piezometer 5-6 Ft.	11 Loss of Head. 1-3 Total length 4 437 Ft.	1-3 Per 1 000 ft. Ft.	3-4 Total length 2 710 Ft.	3-4 Per 1 000 ft. Ft.	5-6 Total length 22 672 Ft.	5-6 Per 1 000 ft. Ft.	4-5 Ft. (44 ft. of wood pipe east of tunnel, + 108.7 ft. of tunnel + 23 ft. of wood pipe west of tunnel.)	12 South Meter. Gauge. Ft.	South Meter. Rate of discharge. Cu. ft. per second.	13 North Meter. Gauge. Ft.	North Meter. Rate of discharge. Cu. ft. per second.	14 Total rate of discharge. Cu. ft. per second.	15 Mean Velocity in Feet per Second. Steel Pipe.	Wood Pipe.	16 No.	17 Gauge.
		−0.004	−0.0009													1	
		0.27	0.060						0.098	19.60			19.60	0.688		2	A.
		0.25	0.057						0.098	19.60			19.60	0.688		4	A.
		0.54	0.121						0.101	31.86			31.86	1.120		5	B.
		0.48	0.109						0.101	31.86			31.86	1.120		6	B.
		0.51	0.121						0.101	31.86			31.86	1.120		7	B.
		0.55	0.124						0.101	31.86			31.86	1.120		8	B.
		0.56	0.127						0.101	31.86			31.86	1.120		9	B.
		5.30	1.197						0.659	80.40	0.188	42.10	122.50	4.341		10	C.
		5.21	1.176						0.666	80.94	0.184	42.67	123.61	4.358		11	C.
		5.23	1.179						0.660	80.46	0.187	42.21	122.67	4.347		12	C.
		5.13	1.157						0.619	79.74	0.188	43.10	122.84	4.318		13	C.
		3.03	0.683						0.555	75.63	0.027	16.53	92.16	3.340		14	D.
		2.90	0.653						0.558	73.94	0.027	16.53	90.47	3.190		15	D.
		2.94	0.664						0.537	73.00	0.027	16.53	89.53	3.135		16	D.
		2.67	0.603						0.510	70.88	0.027	16.53	87.41	3.073		17	D.
		1.31	0.295						0.345	58.46			58.46	2.065		18	E.
		1.20	0.271						0.331	57.27			57.27	2.013		19	E.
		1.20	0.271						0.331	57.27			57.27	2.013		20	E.
		1.31	0.295						0.358	59.53			59.53	2.093		21	E.
		6.08	1.350						0.816	79.30	0.206	45.33	124.63	4.730		22	F.
		6.13	1.381						0.815	79.23	0.206	45.33	124.56	4.730		23	F.
		5.91	1.335						0.812	79.08	0.203	45.00	124.08	4.712		24	F.
		5.73	1.296						0.798	88.04	0.203	45.00	133.04	4.677		25	F.
		1.95	0.440						0.544	73.16			73.16	2.572		26	G.
		1.80	0.406						0.514	73.16			73.16	2.572		27	G.
		1.88	0.425						0.546	73.30			73.30	2.576		28	G.
		1.87	0.421						0.551	73.68			73.68	2.588		29	G.
		5.57	1.256						0.416	64.10	0.308	60.35	124.45	4.375		30	H.
		6.14	1.387						0.505	70.60	0.430	65.88	136.48	4.796		31	H.
		6.15	1.380						0.504	70.48	0.436	65.61	136.07	4.769		32	H.
		6.02	1.359						0.490	70.12	0.431	65.24	135.36	4.758		33	H.
.69	42.49	−0.080	−0.0180	0.00	0.000	+0.040	+0.0018	0.00	0.000	0.00	0.000	0.00	0.00	0.000		34	..
				0.70	0.258				0.088	14.92	0.219	46.73	61.65		2.150	35	I.
.78	44.78			0.73	0.266	4.76	0.210	0.11	0.080	14.24	0.223	47.14	61.38		2.141	36	I.
.75	44.61			0.50	0.184	4.91	0.217	0.12	0.019	13.88	0.240	47.86	61.74		2.154	37	I.
.79	41.73			0.45	0.166	4.79	0.211	0.17	0.019	13.88	0.231	47.24	61.12		2.138	38	I.
	34.98					14.60	0.644		0.387	61.86	0.209	45.65	107.51		3.750	39	..
.78	48.08			0.08	0.030	1.50	0.066	0.16			0.112	33.33	33.33		1.170	40	J.
.75	48.00			0.11	0.041	1.52	0.067	0.13			0.113	33.03	33.03		1.176	41	J.
.80	46.07			0.11	0.041	1.45	0.064	0.20			0.113	33.05	33.05		1.175	42	J.
.72	47.97			0.11	0.041	1.55	0.068	0.10			0.114	33.55	33.55		1.180	43	J.
.87	36.38			1.13	0.417	10.70	0.472	0.25	0.368	53.04	0.114	33.63	86.67		3.290	44	K.
.38	34.74			1.21	0.446	10.78	0.475	0.21	0.354	50.30	0.113	33.03	83.33		2.940	45	K.
.85	35.77			1.18	0.435	10.75	0.474	0.22	0.355	50.30	0.112	32.53	82.82		3.005	46	K.
.88	35.78			1.18	0.435	10.74	0.474	0.21	0.335	53.29	0.112	33.30	86.59		3.095	47	K.
.78	44.85			0.49	0.181	4.67	0.206	0.14	0.018	13.51	0.227	47.55	61.06		2.150	48	L.
.78	44.88			0.53	0.196	4.64	0.205	0.10	0.021	14.30	0.224	47.34	61.64		2.157	49	L.
.71	44.80			0.63	0.231	4.63	0.204	0.09	0.018	13.51	0.219	46.72	60.23		2.101	50	L.
.78	44.86			0.71	0.260	4.70	0.207	0.11	0.019	13.80	0.230	46.91	60.71		2.118	51	L.
.83	36.95			1.30	0.480	11.27	0.497	0.21	0.230	47.76	0.223	47.35	95.11		3.318	52	M.
.84	38.18			1.30	0.480	11.34	0.500	0.22	0.231	47.97	0.230	47.86	95.83		3.343	53	M.
.85	38.15			1.29	0.476	11.37	0.501	0.21	0.230	47.86	0.227	47.55	95.41		3.335	54	M.
.88	38.86			1.25	0.461	11.19	0.494	0.21	0.232	48.07	0.219	46.72	94.79		3.305	55	M.
.75	47.87			0.14	0.052	1.65	0.073	0.12			0.127	35.60	35.60		1.243	56	N.
.73	47.88			0.13	0.048	1.60	0.073	0.11			0.127	35.60	35.60		1.245	57	N.
.71	47.84			0.15	0.055	1.68	0.074	0.09			0.126	35.35	35.35		1.240	58	N.
.67	47.80			0.16	0.059	1.72	0.076	0.05			0.127	35.60	35.60		1.243	59	N.
	34.80					12.63	0.557		0.321	57.27	0.191	43.67	100.94		3.561	60	P.
	36.90					12.63	0.557		0.322	57.66	0.198	43.59	101.25		3.538	61	P.
	36.92					12.40	0.556		0.322	57.44	0.191	43.67	101.11		3.527	62	P.
	37.23					12.30	0.549		0.320	57.19	0.189	42.41	99.60		3.474	63	P.
	40.98					8.56	0.378		0.170	41.32	0.170	42.20	83.52		2.918	64	Q.
	40.96					8.56	0.374		0.170	41.33	0.182	42.44	83.77		2.925	65	Q.
	40.94					8.58	0.378		0.171	41.35	0.180	42.41	83.76		2.922	66	Q.
	40.65					8.97	0.396		0.183	42.76	0.184	42.87	85.63		2.987	67	Q.
	29.33					20.20	0.893		0.490	69.49	0.285	57.61	127.10	4.466	4.426	68	R.
	29.39					20.33	0.897		0.491	69.56	0.342	58.21	127.77	4.491	4.457	69	R.
	29.44					20.10	0.887		0.487	69.28	0.341	58.12	127.40	4.478	4.444	70	R.
	29.64					19.88	0.877		0.484	69.07	0.338	57.70	126.77	4.456	4.420	71	R.
	30.56	7.68	1.730			28.96	1.277		0.563	91.77	0.377	61.07	152.84	5.373	5.351	72	S.
	30.91	7.65	1.727			28.59	1.261		0.557	91.46	0.376	60.99	152.45	5.359	5.317	73	S.
	31.78	7.41	1.675			27.74	1.224		0.518	89.39	0.377	61.07	150.46	5.320	5.245	74	S.
	32.21	7.31	1.651			27.31	1.205		0.504	88.64	0.376	60.99	149.63	5.300	5.219	75	S.
	40.35	+0.004	+0.0009			−0.030	−0.0013		0.000	0.00	0.000	0.0	0.00	0.000	0.000	76	.
.10	26.19			2.69	0.993	23.35	1.030	0.48	0.818	90.98	0.229	47.76	138.75		4.840	77	T.
.08	25.82			2.70	0.996	23.19	1.023	0.42	0.842	90.67	0.229	47.76	138.43		4.880	78	T.
.97	26.16			2.75	1.015	23.36	1.030	0.35	0.850	91.09	0.214	46.27	137.36		4.861	79	T.
.96	26.24			2.73	1.007	23.28	1.027	0.36	0.844	90.78	0.224	46.27	137.05		4.850	80	T.
	27.79					21.78	0.958		0.683	81.86	0.279	52.56	134.42		4.691	81	U.
	27.70					21.84	0.962		0.695	81.94	0.280	52.74	134.68		4.688	82	U.
	27.74					21.78	0.961		0.686	81.94	0.283	53.02	134.96		4.707	83	U.
	27.87					21.65	0.955		0.681	81.71	0.277	52.46	134.17		4.680	84	U.

to the reduced water column. Thus, if the water temperature is t° Cent., the piezometer column, at 10°, computed as above, is multiplied by the factor $1 + 0.000084\ (t - 10)$.

This correction was applied only in case of Manometers Nos. 1 and 2. In the case of the wood pipe it is of comparatively little importance, because of the much smaller slope of the pipe. Moreover, although the flume temperature may give a fairly reliable indication of the variations of temperature in the steel pipe, it would be hopeless to attempt to secure any reliable estimate of the changes of temperature in the long section of wood pipe.

(2.) *Computation of Loss of Head.*—The loss of head between any two stations, due to a given velocity of flow, may be found by determining the difference between the piezometer columns at the two stations, and comparing it with the like difference under static conditions.* Thus, if z denotes the difference between simultaneous values of the piezometer heights at the two stations, and Z is the value of z under static conditions, $Z - z$ is the loss of head between the two stations. The datum of reference must remain constant for each piezometer, but its actual position is arbitrary. If the two sections of pipe have unequal diameters, the velocity-head must be added to the piezometer height in every case; but, with a uniform pipe, the velocity terms disappear from the value of the loss of head.

(3.) *Computation of Rate of Discharge.*—From a difference-gauge observation, the rate of discharge is computed in the usual way. The velocity through the throat of the Venturi is given by the formula $v = k' \sqrt{2\, g\, H}$, in which H is "head on Venturi," and the coefficient k' is the product of the friction coefficient k (in the notation of the previous paper), and the coefficient depending upon the areas of the upper section and throat of the Venturi. The values of k' for the Ogden meters, as furnished by the manufacturers were given in the former paper.†

General Results.

From each observation-group, the values of the following quantities have been computed: Loss of head per thousand feet; coefficient c in the Chezy formula $v = c \sqrt{r\, s}$; and "coefficient of roughness"

* In the previous paper (*Transactions*, Am. Soc. C. E., Vol. xl, p. 481), the method used was substantially the same; but the loss of head was first computed in mercury and then reduced to water, while in the present case each separate manometer observation was reduced to an equivalent water piezometer reading. This method was adopted for the reason that one of the gauges was a water piezometer, and its readings required no reduction.

† *Transactions*, Am. Soc. C. E., Vol. xl, p. 483.

n in Kutter's formula. These have been tabulated, and the values of c, n and loss of head per thousand feet are also shown graphically, being plotted with mean velocity of flow v as abscissas. Mean curves are drawn, representing each of these quantities as a function of the velocity, and, from the mean curves, generalized tables are computed. In these tables are included values of the friction coefficient f in the formula

$$H' = f \frac{l}{d} \frac{v^2}{2g},$$

H' being total loss of head in length l of pipe. These values are computed from those of c, from the relation

$$f = \frac{8g}{c^2}.$$

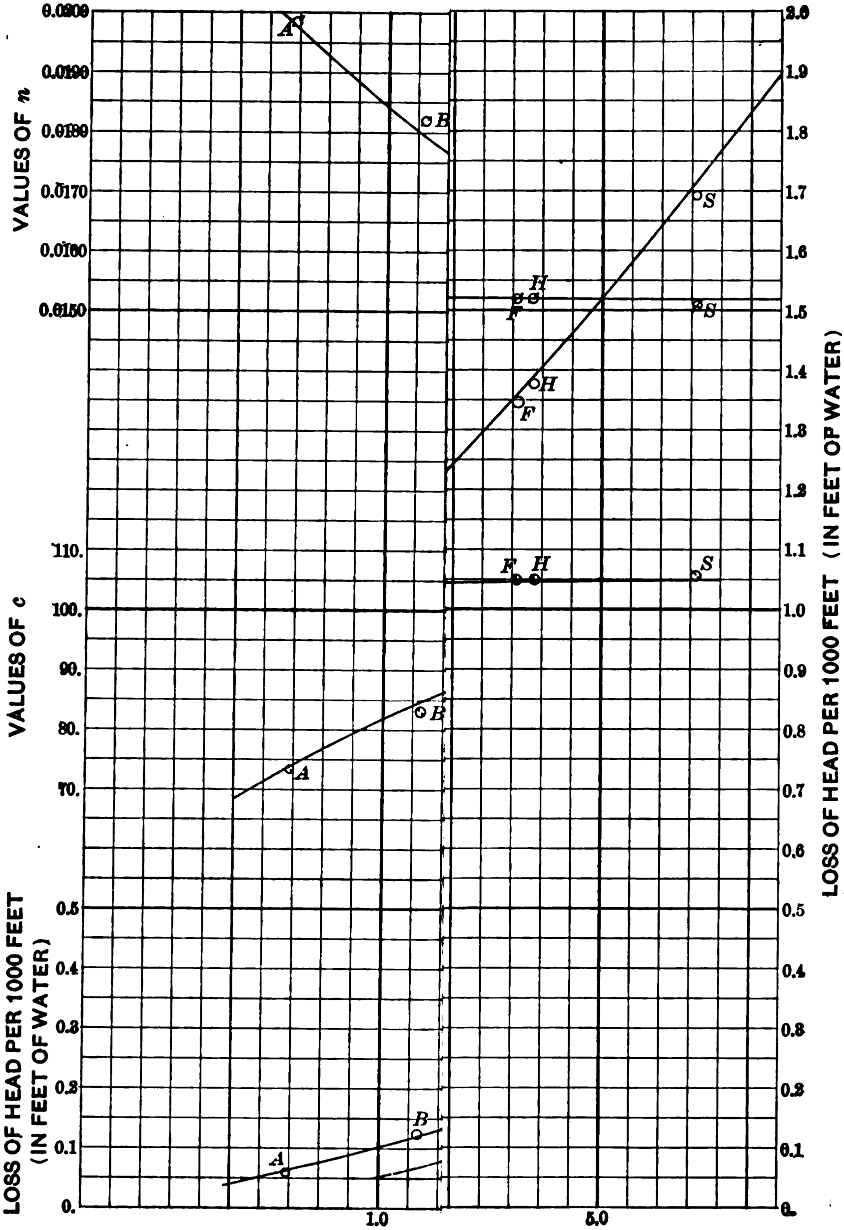
II.—Results of the Experiments.

Record of Observations.

The experimental data for the entire series of observations are shown in Table No. 1. The observations are numbered in chronological order, the numbers being given in Column 2 of the table. The total number is 84; but No. 3 was rejected because it showed so great a discrepancy when compared with Nos. 2 and 4, in which the rate of discharge had the same value as in No. 3. In nearly every case the period of an observation was 20 minutes, though the actual interval during which readings were taken was less than this by from 2 to 5 minutes, because of the time occupied in opening the blow-off valves at the beginning of each period. The date and time of each observation are given in Column 3.

Columns 4, 5, 6, 7 and 8 contain the data obtained from the five mercury gauges at Stations Nos. 1, 2, 3, 4 and 5. In each case, the following quantities are entered: Upper scale reading (U), lower scale reading (L), temperature of mercury column (T). In the case of Manometers Nos. 1 and 2, the temperature of the water in the steel pipe (t) is also given. The constant distance between zeros of upper and lower scales is in each case entered at the top of the column. The sub-column headed "Mer." gives the actual height of the mercury column, and the sub-column "Mer. Reduc." gives the height of the mercury column corrected for temperature. In the case of Manometers Nos. 1 and 2 there is a double temperature correction, account being taken of both mercury and water temperatures.

PLATE XVI.
PAPERS AM. SOC. C. E.
FEBRUARY, 1900.
K, WING AND HOSKINS ON FLOW OF WATER.



The method of applying the temperature correction may be explained by reference to a single observation, as No. 18. The data entered for Manometer No. 1 are as follows: Upper reading, 2.214; lower reading, 0.033; constant distance between zeros of scales, 31.300; mercury temperature, 10.8° Cent.; water temperature, 11.5° Cent. The actual height of the mercury column is, therefore, $31.300 + 2.214 - 0.033 = 33.481$ ft. To reduce to an equivalent column at 10° Cent. there must be subtracted $33.481 \times 0.00018 \times 0.8 = 0.005$ ft., 0.00018 being the coefficient of expansion of mercury. The correction for water temperature, already explained, has for convenience been applied to the mercury column before reduction to equivalent water column. In Observation No. 18 the temperature of the water in the steel pipe is 1.5° higher than the standard temperature of 10° Cent. The corresponding correction to the mercury column is $33.481 \times 0.000084 \times 1.5 = 0.004$ ft., 0.000084 being taken as the coefficient of expansion of water at temperatures near 10° Cent. The correction for mercury temperature is negative, while that for water temperature is positive; hence the reduced height of mercury column ("Mer. Reduc.") is $33.481 + 0.004 - 0.005 = 33.480$ ft.

It should be remarked that the constant distance between zeros need not be determined with great accuracy. Its only importance is in the computation of the temperature correction.

In Column 9 is given, for each gauge, the height of the "equivalent water piezometer." The datum of reference for each gauge is, as already remarked, arbitrary. If the zero of the lower scale is taken as datum, the equivalent water piezometer height is determined by multiplying the reduced mercury column by 13.57 (the relative specific gravity of the mercury at 10° Cent.), and adding the lower scale reading. Thus, referring to Observation No. 48, the data for Manometer No. 3 are

7.136 = reduced height of mercury column;

0.006 = lower scale reading.

Hence the height of the equivalent water piezometer is $7.136 \times 13.57 + 0.006 = 96.84$ ft.

In the case of Manometer No. 1, the reduced mercury column is in every case diminished by 30 ft. before the reduction to "equivalen water piezometer." This simplifies the logarithmic computation, and is allowable because it amounts merely to shifting the datum of refer-

ence by 407.10 ft. (the water equivalent of 30 ft. of mercury). The values of "Mer. Reduc." for Gauge No. 2 have in like manner been diminished by 8 ft.

The sixth sub-column under Column 9 gives the reading of Gauge No. 6. The "equivalent water piezometer" is in this case the actual reading of the gauge. Since the height of water column is only 5 or 6 ft., the temperature correction for this gauge is inappreciable. About 20° change of temperature of the column would be required to change the reading by 0.01 ft.

Sub-column 7, under Column 9, gives the stage of water in the reservoir above the dam. It is obtained by measuring down from a fixed point on the masonry, and the values are therefore given the minus sign. Many of the values entered are obtained by interpolation between observations made an hour or more apart. No special device was used to secure great accuracy in these readings, and in some cases they are affected by an uncertainty of perhaps 0.05 ft. because of waves. These readings are not used in estimating pipe coefficients, but were taken for the purpose of estimating the loss of head in the entire pipe line.

For computing the loss of head between two manometer stations, the difference between simultaneous piezometer columns at these stations is taken. Values of this difference for each of the three lengths of pipe experimented on, and also for Gauges Nos. 4 and 5, are given in Column 10. These values are found in each case by taking differences between the numbers in the corresponding sub-columns of Column 9. The process of computing loss of head is completed by subtracting these differences from the corresponding static difference for each pair of manometers compared. The results are given in Column 11.

Static readings could be secured only during one hour each week, when the water-wheels were stopped. Three sets of static readings were taken, the results being recorded as Observations Nos. 1, 34 and 76. When Observation No. 1 was made, only Manometers Nos. 1 and 2 were ready for use. Observation No. 34 includes readings of all manometers. In Observation No. 76 readings were taken on Manometers Nos. 1, 2, 5 and 6. The static difference for the steel pipe (1–2) was thus observed three times; but as Observation No. 34 on the steel pipe was not regarded as very satisfactory at the time, the static dif-

PLATE XVII.
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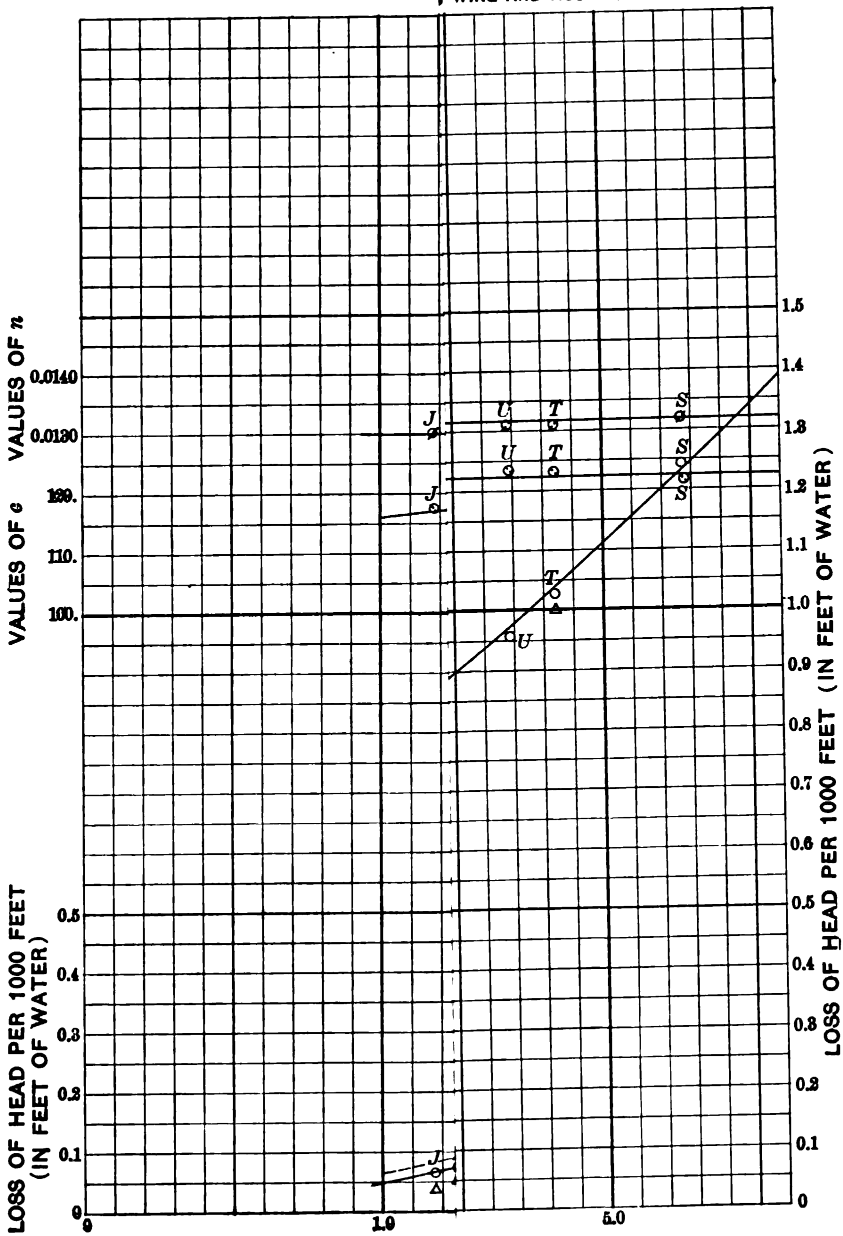
WING AND HOSKINS ON FLOW OF WATER.



ference adopted for Gauges Nos. 1 and 2 was based upon Observation Nos. 1 and 76, which differ by only 0.01 ft. The two values are 13.19 and 13.18; when the computation is carried to the third decimal place the mean of these is 13.186, and value adopted is 13.19. It will be seen that Observation No. 34 gives a value differing from this by 0.06 ft. If this had been used in computing the mean, the result would have been changed by only 0.02 ft. Only one determination of the static difference 3–4 was made. For 5–6 there were two determinations, giving values 49.49 and 49.55, the mean of which (49.52) was adopted.

In Column 11 are entered the values of the total loss of head and of the loss per thousand feet, for each of the three lengths of pipe 1–2, 3–4, 5–6. At the top of each sub-column is given the length of pipe between manometers. A sub-column is also given for values of the loss between Gauges Nos. 4 and 5, from which the loss due to Tunnel No. 7 may be estimated.

In Columns 12, 13, 14 and 15 are entered the results of the discharge observations. The sub-column headed "gauge" for each meter gives the difference between the heights of the throat and up-stream mercury columns of the Venturi meter difference-gauges. "Head on Venturi" is equal to this difference multiplied by $e-1$, e being the specific gravity of the mercury, 13.57. From this gauge reading and the known dimensions of the meter the rate of discharge is computed, as already explained. The remaining columns, giving rate of discharge for each meter, total rate of discharge, and mean velocity of flow in the steel and in the wood pipe, need no explanation.

In most cases several consecutive observations (usually four) were made with as nearly constant conditions of discharge as possible. When the actual variation was small, such observations were combined into a "group." These groups are designated by letters, which are given in Columns 1 and 17 of Table No. 1. Table No. 2 shows values of mean velocity and loss of head, as obtained by averaging the results of each group of observations. The other quantities given in this table will be referred to later.

Steel Pipe Results.

The steel pipe results for each observation-group are given in Table No. 2, Columns 3, 4, 5 and 6.

TABLE No. 2.

1	2	3	4	5	6	7	8	9	10	11	12	13
	STEEL PIPE. MAN. 1-2.					WOOD PIPE. MAN. 3-4.		WOOD PIPE. MAN. 5-6.				TOTAL LOSS. MAN. 4-5.
Group.	Numbers of observations.	Velocity in feet per second.	Loss of head per 1 000 ft. in feet.	c.	n.	Velocity in feet per second.	Loss of head per 1 000 ft. in feet.	Velocity in feet per second.	Loss of head per 1 000 ft. in feet.	c.	n.	Total loss in feet.
	1	0.	-0.009	......	......	......	......	......	......	......	......	......
A..	2, 4	0.689	0.059	73.4	0.0198	......	......	......	......	......	......	......
B..	5, 6, 7, 8, 9	1.120	0.121	88.1	0.0182	......	......	......	......	......	......	......
C..	10, 11, 12, 13	4.339	1.177	103.1	0.0153	......	......	......	......	......	......	......
D..	14, 15, 16, 17	3.157	0.645	101.3	0.0157	......	......	......	......	......	......	......
E...	18, 19, 20, 21	2.043	0.286	98.5	0.0175	......	......	......	......	......	......	......
F...	22, 23, 24, 25	4.718	1.343	104.8	0.0152	......	......	......	......	......	......	......
G...	26, 27, 28, 29	2.577	0.423	102.1	0.0154	......	......	......	......	......	......	......
	30	4.375	1.258	100.6	0.0158	......	......	......	......	......	......	......
H..	31, 32, 33	4.779	1.379	104.9	0.0152	......	......	......	......	......	......	......
	34	0.000	-0.000	......	......	0.000	0.000	0.000	0.001	......	......	......
I...	35, 36, 37, 38	......	......	......	......	2.144	0.219	2.144	0.218	119.5	0.0132	0.137
	39	......	......	......	......	3.750	......	3.750	0.644	120.2	0.0133	......
J...	40, 41, 42, 43	......	......	......	......	1.175	0.038	1.175	0.066	117.3	0.0130	0.147
K..	44, 45, 46, 47	......	......	......	......	3.239	0.434	3.239	0.474	121.1	0.0132	0.225
L..	48, 49, 50, 51	......	......	......	......	2.126	0.221	2.126	0.206	120.7	0.0130	0.110
M..	52, 53, 54, 55	......	......	......	......	3.324	0.474	3.324	0.498	121.2	0.0132	0.212
N..	56, 57, 58, 59	......	......	......	......	1.244	0.054	1.244	0.074	117.7	0.0130	0.095
P..	60, 61, 62	......	......	......	......	3.527	......	3.527	0.557	121.7	0.0132	......
	63	......	......	......	......	3.474	......	3.474	0.542	121.4	0.0132	......
Q..	64, 65, 66	......	......	......	......	2.920	......	2.920	0.378	122.2	0.0131	......
	67	......	......	......	......	2.987	......	2.987	0.396	122.2	0.0131	......
R..	68, 69, 70, 71	4.478	......	......	......	4.439	......	4.439	0.889	121.2	0.0132	......
S...	72, 73, 74, 75	5.320	1.696	105.3	0.0151	5.279	......	5.279	1.242	121.9	0.0132	......
	76	0.000	+0.009	......	......	0.000	......	0.000	-0.001	......	......	......
T..	77, 78, 79, 80	......	......	......	......	4.845	1.008	4.845	1.027	123.0	0.0131	0.402
U.	81, 82, 83, 84	......	......	......	......	4.694	......	4.694	0.959	123.3	0.0131	......

The same results are shown graphically in Plate XVI. In this diagram mean curves are drawn to represent the relation between mean velocity of flow and each of the three quantities, loss of head per thousand feet, *c* and *n*. The curve of loss of head given by the experiments of 1897 is also shown for the purpose of comparison. The later series covers a range of velocities materially greater than the earlier, the greatest value of the mean velocity being 5.32 ft. per second in the observations of 1899 as against 3.85 ft. per second in the series of 1897.

A reference to Fig. 6 of the previous paper * shows that the portion of the curve corresponding to the higher velocities was based largely upon five observations made under conditions of falling press-

* *Transactions*, Am. Soc. C. E., Vol. xl., p. 495.

TABLE No. 3.—Steel Pipe. Generalized Results.

v.	H'.		c.		f.		n.	
	1897.	1899.	1897.	1899.	1897.	1899.	1897.	1899.
1.0	0.055	0.100	110.0	81.6	0.0212	0.0387	0.0133	0.0184
1.5	0.121	0.177	111.0	92.0	0.0207	0.0303	0.0140	0.0167
2.0	0.220	0.277	110.0	98.0	0.0212	0.0267	0.0144	0.0159
2.5	0.356	0.405	108.0	101.3	0.0221	0.0251	0.0147	0.0155
3.0	0.510	0.570	108.0	102.4	0.0221	0.0245	0.0147	0.0154
3.5	0.673	0.765	110.0	103.2	0.0212	0.0242	0.0145	0.0153
4.0	0.863	0.987	111.0	103.8	0.0207	0.0238	0.0143	0.0153
4.5	....	1.237	....	104.3	....	0.0236	....	0.0152
5.0	....	1.516	....	104.7	....	0.0234	....	0.0152
5.5	....	1.824	....	105.0	....	0.0233	....	0.0152

ure. It seems possible, in the light of the later results, that this portion of the curve is somewhat too low. But making all reasonable allowance for the uncertainty in the values found, it appears that there has been some decrease in the carrying capacity of the steel pipe.

By interpolation, using the mean curves of Plate XVI, values of the several quantities may be found for velocities 1, 1.5, etc., feet per second. These generalized values of H', c, f and n are shown in Table No. 3, together with the corresponding values from the observations of 1897.

The Chezy coefficient c shows an increase with the velocity. For velocities above 2.5 ft. per second, this increase, however, is slow, and the upper limit to the value of c would appear to be not greater than 106.

The Kutter coefficient n shows, for low velocities, a decrease with increasing velocity. For higher velocities n approaches a limiting value of about 0.0152.

It must be remembered that the law of variation of c and of n for low velocities cannot be regarded as reliable, because the values found for these quantities are affected very materially by small errors in the measured loss of head.

Wood Pipe Results.

The wood pipe results for all the observation-groups are shown in Table No. 2. For the shorter length of pipe (3–4) the table gives only the velocity and the loss of head per thousand feet (Columns 7 and 8). For the longer portion, values of c and n are also given.

TABLE No. 4.—WOOD PIPE. GENERALIZED RESULTS.

v.	H'.		c.		f.		n.	
	1897.	1899.	1897.	1899.	1897.	1899.	1897.	1899.
1.0...........	0.066	0.049	100.0	116.0	0.0257	0.0191	0.0150	0.0130
1.5...........	0.123	0.106	110.0	118.7	0.0212	0.0183	0.0141	0.0130
2.0...........	0.200	0.184	115.0	119.9	0.0194	0.0179	0.0137	0.0131
2.5...........	0.292	0.284	119.0	120.8	0.0181	0.0176	0.0133	0.0132
3.0...........	0.400	0.404	122.0	121.4	0.0173	0.0175	0.0131	0.0132
3.5...........	0.527	0.548	124.0	121.7	0.0167	0.0174	0.0130	0.0132
4.0...........	0.678	0.712	125.0	122.0	0.0165	0.0172	0.0128	0.0132
4.5...........	....	0.898	....	122.2	....	0.0172	....	0.0132
5.0...........	....	1.105	....	122.4	....	0.0172	....	0.0132
5.5...........	....	1.335	....	122.5	....	0.0171	....	0.0132

The coefficients c and n and the loss of head per thousand feet are shown graphically in Plate XVII, being plotted as functions of the mean velocity of flow. This diagram shows the observations on both portions of the wood pipe, the two sets of points being marked differently.

As the number of observers was too small to permit the simultaneous occupation of all the six manometer stations, it was thought best to devote special attention to the long section of wood pipe, and to the steel pipe. The number of observations upon the shorter length of wood pipe was therefore limited, their main value being to serve as a check upon the results of 1897, and to show whether any material difference exists between the coefficients for the two portions of wood pipe. The mean curves shown in Plate XVII are intended to represent the results for the long section only. For the purpose of comparing with the previous experiments on the lower section, the mean curve of loss of head as found from the experiments of 1897 is also shown.

The generalized results for the upper section of wood pipe are given in Table No. 4, the values of H', c, f and n being obtained from the mean curves of Plate XVII. The table includes also values of the same quantities for the lower section of pipe, as found from the experiments of 1897.

Loss of Head Due to Tunnel No. 7.

Tunnel No. 7 is 108.7 ft. long, and is unlined, its cross-section being approximately 9 ft. square. Manometer Stations Nos. 4 and 5

were on opposite sides of this tunnel, the former being 23 ft. from its west end and the latter 44 ft. from its east end. Simultaneous readings of the gauges at these two stations furnish date for estimating the loss of head due to the tunnel.

TABLE No. 5.—Tunnel 108.7 Ft. Long. 9 Ft. Square.

Velocity in wood pipe, in feet per second.	Loss between manometers 4 and 5, in feet.	Loss in 67 ft. of wood pipe, in feet.	Loss due to tunnel, in feet.	Equivalent length of wood pipe, in feet.
1.0	0.044	0.003	0.041	885
2.0	0.110	0.012	0.098	532
3.0	0.190	0.027	0.163	405
4.0	0.290	0.048	0.242	340
5.0	0.430	0.074	0.356	322

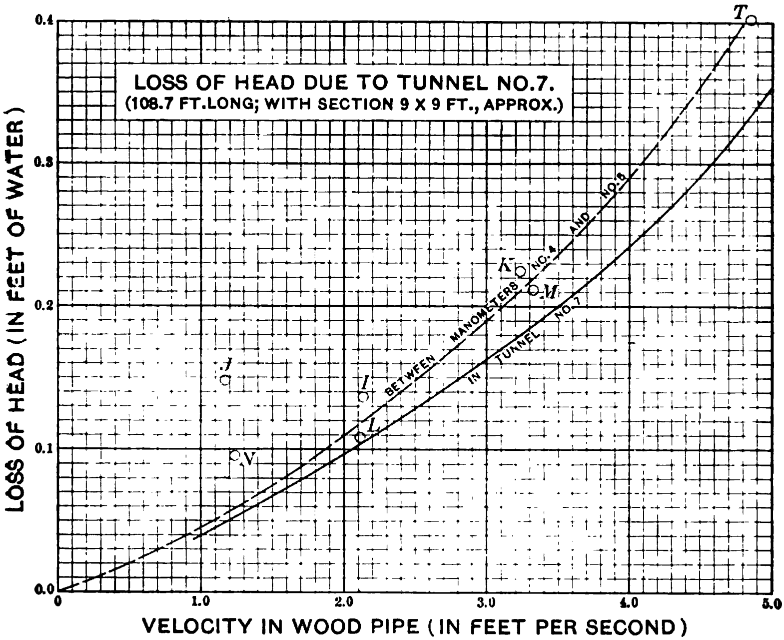


Fig. 3.

The total loss between Gauges Nos. 4 and 5 is given in Table No. 1, Column 11, and the condensed results are shown in Table No. 2, Column 13. These results are shown graphically in Fig. 3, in which is also drawn a curve to represent the relation between the total loss

TABLE No. 6.

1	2	3	4	5	6	7	8	9	10
No.	South Meter.		North Meter.		Group.	South Meter.		North Meter.	
	$\frac{H}{e-1}$	$\frac{H''}{e-1}$	$\frac{H}{e-1}$	$\frac{H''}{e-1}$		$\frac{H}{e-1}$	$\frac{H''}{e-1}$	$\frac{H}{e-1}$	$\frac{H''}{e-1}$
2	0.088	0.006			*A*				
3	0.088	0.007			*A*	0.088	0.007		
4	0.088	0.007			*A*				
5	0.101	0.020			*B*				
6	0.101	0.019			*B*				
7	0.101	0.019			*B*	0.101	0.019		
8	0.101	0.019			*B*				
9	0.101	0.019			*B*				
10	0.659	0.112	0.186	0.080	*C*				
11	0.668	0.118	0.184	0.080	*C*	0.659	0.111	0.186	0.080
12	0.660	0.112	0.187	0.081	*C*				
13	0.648	0.108	0.186	0.080	*C*				
14	0.582	0.099	0.027	0.008	*D*				
15	0.556	0.094	0.027	0.008	*D*	0.546	0.091	0.027	0.008
16	0.537	0.090	0.027	0.008	*D*				
17	0.510	0.082	0.027	0.008	*D*				
18	0.345	0.058			*E*				
19	0.331	0.056			*E*	0.341	0.058		
20	0.331	0.056			*E*				
21	0.358	0.061			*E*				
22	0.816	0.188			*F*				
23	0.815	0.187			*F*	0.818	0.187		
24	0.812	0.186			*F*				
26	0.544	0.091			*G*				
27	0.544	0.091			*G*	0.545	0.092		
28	0.546	0.095			*G*				
31	0.506	0.087			*H*	0.506	0.087		

of head between gauges and the mean velocity of flow in the wood pipe. Upon this curve are based the generalized results given in Table No. 5.

The total length of wood pipe between the two gauges is 67 ft.; the loss of head due to this length, assuming the loss per thousand feet to have values as given in Table No. 4, is computed and entered in Table No. 5. Deducting this from the total loss between gauges, the loss due to the tunnel is found. The total tunnel loss is made up of loss at entrance, loss at outlet, and loss due to resistances in the tunnel itself. It is not possible to estimate these losses separately so as to determine the value of *c* for the tunnel. The curve showing total loss due to tunnel for different values of velocity in the wood pipe is given in Fig. 3.

The last column of Table No. 5 shows the length of wood pipe which would produce the same loss as the tunnel.

Loss of Head in Venturi Meters.

A limited number of observations was made on the loss of head due to the Venturi meters. The first and third tubes of each of the triple difference-gauges communicated with the pipe at points where the values of the mean velocity were equal, the diameter being 54 ins. at each section. The observed pressure-difference between these sections, therefore, was due wholly to the loss of head in the intervening portion of the stream.

It should be stated that this observed loss is not all chargeable to the meter. The portion of the pipe between the two pressure sections included, besides the converging and diverging portions of the meter, about 7 ft. of 54-in. riveted pipe. The diverging pipe below the throat is constructed of riveted plates, and a gate-valve, occupying

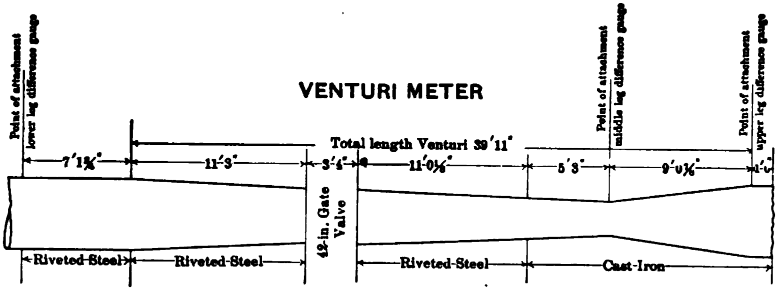


FIG. 4.

about 40 ins., is situated in this portion of the pipe. The whole arrangement is shown in Fig. 4. Doubtless, the greater part of the observed loss of head occurred between the throat and the down-stream pressure section. While the loss in the diverging pipe may be properly charged to the meter, since this is a necessary part of the apparatus, this is not true of the loss in the 7 ft. of 54-in. pipe below, nor of the loss due to the gate. The points of attachment of the difference-gauge tubes were the same in these experiments as in the previous series.

The results of the observations on the loss of head in the meters are shown in Table No. 6. "Head on Venturi" is denoted by H, and "loss of head" by H''. The specific gravity of the mercury being e, the difference between the up-stream and throat mercury columns is

$\frac{H}{e-1}$, and the difference between the up-stream and down-stream columns is $\frac{H''}{e-1}$. These are the quantities observed directly, and the observed values are given in the table. These values are averaged in groups, and the average values are entered in Columns 7, 8, 9 and 10. The observation numbers and the letters designating groups agree with those in Table No. 1.

The results are plotted in Fig. 5. As in the previous series of observations,* the plotted points fall very nearly on a straight line, indicating a relation expressed by the equation $H'' = a\,H$, with a constant value of a. The value found for a is 0.169, while in 1897 the value was 0.149. The lines corresponding to both these values are shown in Fig. 5.

It thus appears that the loss of head between the two sections compared increased 13.4% between August, 1897, and June, 1899. This is on the assumption that the loss of head in the meter proper (*i. e.*, between the upper section and the throat) has remained constant; for unless this is true a given value of "head on Venturi" corresponds to different values of the rate of discharge in the two series of experiments.

How nearly correct this assumption is, there are no means of knowing. It is probable, however, that the change in the loss in the converging part of the meter is slight.

It may be pointed out that an increase of 13.4% in the loss of head, for any given velocity, agrees well with the increase observed in the 72-in. steel pipe. This may be verified by a comparison of the 1897 and 1899 curves of loss of head in Plate XVI.

The continued courtesy of C. K. Bannister, M. Am. Soc. C. E., Chief Engineer of the Union Power Company, and of the Board of Directors, enabled the authors to duplicate and extend the series of observations made in 1897. To them and to Mr. L. S. Boggs, Electrical Engineer, the authors are under great obligations for giving *carte blanche*, so far as they were able to do so without interfering with the running of the plant. As the authors were short-handed at times, the work could not have been carried through in the time at their disposal if the Superintendent of the Power Company, Mr. C. E. Crocker,

* See Figs. 13 and 14 of previous paper.

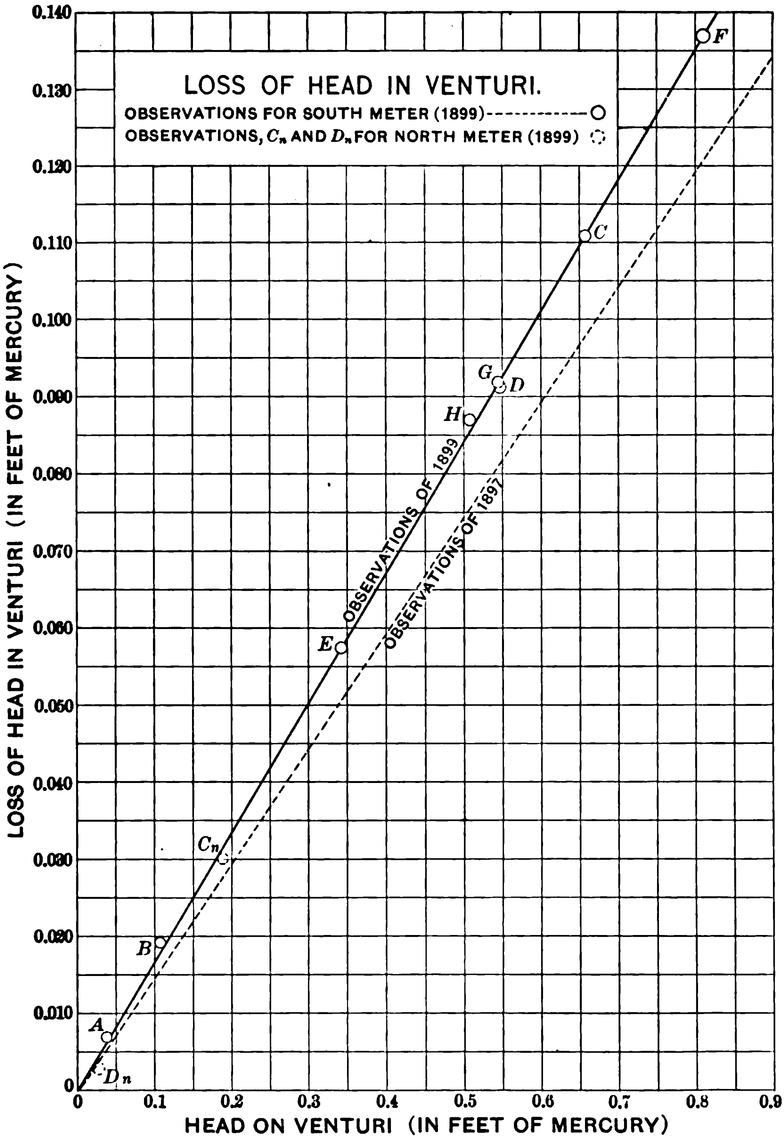

LOSS OF HEAD IN VENTURI.
OBSERVATIONS FOR SOUTH METER (1899)
OBSERVATIONS, C_n AND D_n FOR NORTH METER (1899)
LOSS OF HEAD IN VENTURI (IN FEET OF MERCURY)
HEAD ON VENTURI (IN FEET OF MERCURY)
OBSERVATIONS OF 1899
OBSERVATIONS OF 1897
0.140
0.130
0.120
0.110
0.100
0.090
0.080
0.070
0.060
0.050
0.040
0.030
0.020
0.010
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
A
B
C_n
D_n
E
H
G
D
C
F

FIG. 5.

had not kindly assisted in taking readings. He also regulated the discharge so as to give, so far as practicable, the desired velocities. For his assistance and co-operation the authors desire to express their sincere thanks. Their former assistant, Mr. L. B. Spencer, Assistant Engineer, Oregon Short Line, came from Salt Lake on two occasions to help, and to him also the authors owe thanks. During the entire period of their stay they were, furthermore, assisted by their colleague, Professor J. C. L. Fish, who has added to their obligations by preparing a number of the diagrams which accompany this paper.

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PAPERS AND DISCUSSIONS.

HISTORY OF THE PENNSYLVANIA AVENUE SUBWAY, PHILADELPHIA, AND SEWER CONSTRUCTION CONNECTED THEREWITH.

By GEORGE S. WEBSTER and SAMUEL TOBIAS WAGNER, Members, Am. Soc. C. E.

TO BE PRESENTED MARCH 21ST, 1900.

It is the purpose of this paper to present the more interesting features of the history of the work of abolishing grade crossings on Pennsylvania Ave., in the City of Philadelphia, by what is known as the Pennsylvania Avenue Subway, and also the construction of the sewers connected therewith.

GENERAL DESCRIPTION.

The Philadelphia and Reading Railroad enters the densely populated portion of the City of Philadelphia by means of two principal systems:

First.—That from Wayne Junction, Tioga, and Ninth St. to the Reading Terminal at Twelfth and Market Sts., formerly on the New York Division.

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

Second.—That from Falls of Schuylkill, Belmont and Pennsylvania Ave., also connecting with the Reading Terminal near Twelfth and Callowhill Sts., formerly on the Main Line Division.

Both of these systems are now part of the Philadelphia Division.

In 1893 both of these lines contained a very large number of grade crossings, which were becoming more dangerous as the city grew and the population on the lines of the tracks increased.

On account of the large number of tracks which formerly crossed Broad St. at grade between Callowhill St. and Pennsylvania Ave., and because Broad St. is one of the widest and finest streets in the city, being, at the point referred to, on one of the principal routes from the center of the city to Fairmount Park, special attention had been directed for some years to an arrangement between the City of Philadelphia and the Philadelphia and Reading Railroad Company by means of which this particular grade crossing could be avoided.

On December 26th, 1890, an ordinance was approved by the Mayor, giving the Philadelphia and Reading Terminal Railroad Company the right to construct the Terminal Station at Twelfth and Market Sts., and connect by means of an elevated structure with the tracks of the Philadelphia and Reading Railroad Company at:

First.—A point near Broad St. and Pennsylvania Ave., and,

Second.—A point near Ninth St. and Fairmount Ave.

In consideration of this privilege the Philadelphia and Reading Railroad Company was, by means of changes of the grades of the streets, to abolish the grade crossings at Broad St. and Lehigh Ave., at Ninth St. and Columbia Ave., and at Broad St. and Pennsylvania Ave., all without expense to the city. By this ordinance, Broad St. was to be carried over the railroad at both Pennsylvania and Lehigh Aves. The plans for this change at Broad St. and Pennsylvania Ave. provided for raising Broad St. about 21 ft., and was known as the "Hump" plan.

After the construction of the bridge, with its approaches at Lehigh Ave., in the northern section of the city, it was demonstrated clearly that a similar raising of the grade at Pennsylvania Ave., almost in the heart of the city, would not only seriously damage valuable property, but would practically ruin the finest avenue in the city, and would leave sixteen dangerous grade crossings to the westward unprovided for.

Before arrangements were made to begin work at Pennsylvania Ave., several plans were prepared by different interests, with a view of avoiding the hump on Broad St. One of these contemplated the abolishment of all grade crossings on the line of Pennsylvania Ave. and Noble St. from Thirteenth to Thirtieth Sts. by means of a tunnel extending from Hamilton St. to a point near Taney St., and the remainder of the line, including the freight yards of the railroad company, in an open subway with bridges on the line of Broad and other cross streets. This plan contemplated but a slight change in the grade of Broad St., and had the additional advantage of abolishing sixteen other dangerous grade crossings over Pennsylvania Ave., besides providing an entrance to Fairmount Park free from all danger and annoyance from passing trains.

The question of elevating the railroad tracks on the avenue was considered, but as this type of construction would result in all the intersecting streets being crossed by bridges, over which constant and heavy traffic would be passing, and particularly as, in addition to Broad St., these crossings would be over Twenty-second, Spring Garden and Green Sts., and Fairmount Ave., all principal entrances to Fairmount Park, the scheme was objected to seriously by citizens in the central and southern portions of the city. The bridge proposed at Broad St. was specially protested against as, on account of the crossing being in the center of the railroad terminal at this point, the width of the overhead bridge would have been 265 ft., thus practically making a tunnel of this length over the street, and cutting off the view on the central boulevard of the city.

After the preparation of a number of studies by the engineers of the city, of the Philadelphia and Reading Railroad, and by the late John A. Wilson, M. Am. Soc. C. E., Consulting Engineer for the Railroad Company, a plan and estimates were prepared for depressing the tracks and carrying them in subway and tunnel beneath all the streets from and including Broad St. to the west as far as Thirtieth St., permanently abolishing all the grade crossings, providing for the railroad business and for the maintenance of adequate track connections along the avenue from Thirteenth to Twenty-second Sts.

The advantages of a subway are that it does not disfigure the streets which cross it, the grades being so arranged that the superstructures of the bridges are beneath the level of the streets, and the

contour therefore remains unbroken. By means of the tunnel from Twenty-second St. to the west, the tracks are removed from sight, and what was formerly a half-blighted section of the city will be reclaimed, obtaining for the city a splendid driveway and an appropriate entrance to Fairmount Park freed from all objectionable features. It will also result in a partial financial return in increased taxes. From a railroad standpoint, it will do away with the constant expense of watchmen, the maintenance of safety gates, and will allow of the rapid running of trains on a clear permanent way, besides practically doing away with all claims arising from accidents to the public.

Legislation.

In the spring of 1894, a printed draft of an ordinance for the work generally described above, together with lithographs of bird's-eye views of Pennsylvania Ave. as it existed at that time and as it was proposed to change it, was placed in the hands of Common and Select Councils, the Legislative bodies of the city, together with an estimate of the total cost of the work, which was placed at $6 000 000. These lithographs are reproduced in Figs. 1 and 2 and show clearly the intent of the work to the average layman.

On March 15th, 1894, an ordinance was approved by the Mayor, then the Honorable Edwin S. Stuart, providing for the issuance of a loan of $6 000 000 to be used for the work. The ordinance provides that the city shall negotiate the loan, as specially provided, and the railroad company shall eventually pay back to the city half the cost of the work, providing that their share of the said cost in no event exceeds the sum of $3 000 000. The loan to be issued in twenty series of $300 000 each.

On March 17th, 1894, an ordinance was approved by the Mayor, for carrying out the work in substantial accordance with the general plans previously mentioned.

The work authorized by this ordinance involved the depression of the tracks and yards of the Philadelphia and Reading Railroad Company between Broad and Thirtieth Sts., including the alteration of the lines and grades of the tracks and yards between the north side of Noble St. and Callowhill St., and between Eleventh and Broad Sts.; the alteration of the lines and grades of the tracks of the Philadelphia

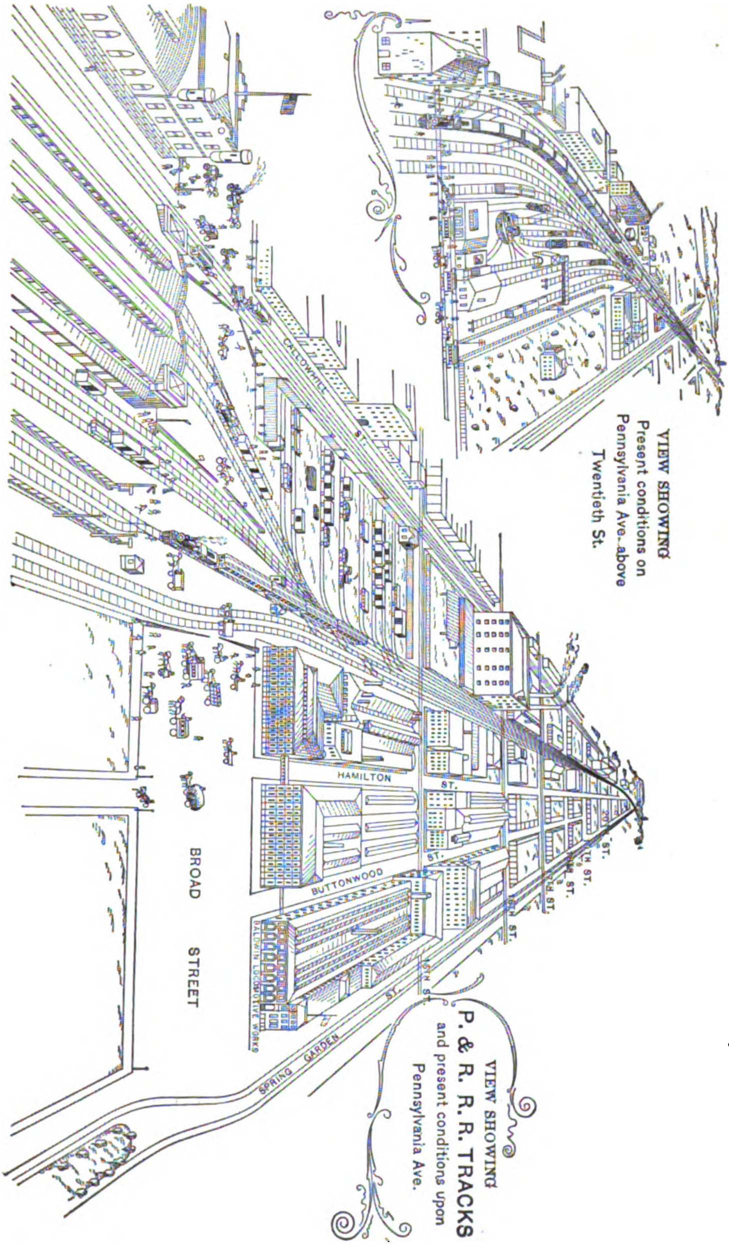

VIEW SHOWING
Present conditions on
Pennsylvania Ave. above
Twentieth St.
VIEW SHOWING
P. & R. R. R. TRACKS
and present conditions upon
Pennsylvania Ave.
HAMILTON ST.
BUTTONWOOD ST.
BROAD STREET
SPRING GARDEN ST.
CALLOWHILL
BALDWIN LOCOMOTIVE WORKS

FIG. 1.

and Reading Terminal Railroad Company, east of Broad St., and between Noble and Carlton Sts. Accompanying this depression and alteration of tracks and yards, is the alteration, construction and reconstruction of all the yard tracks, freight, engine, depot and signal buildings and other structures of the Philadelphia and Reading Railroad Company, so as to provide as much accommodation and as full and convenient a method for operation and conducting business as previously existing. Adequate track connections with the various industrial establishments along the line of the railroad were to be provided; provision was to be made for the construction of temporary tracks and bridges for maintaining travel on the railroad and upon all the intersecting streets. Bridges with adequate approaches, piers and abutments were to be built to carry Broad St., Fifteenth, Sixteenth, Seventeenth, Eighteenth, Nineteenth, Twentieth and Twenty-first Sts. over the tracks of the railroad. Bridges at Callowhill, Twelfth and Thirteenth Sts. were to be built to carry the tracks of the railroad over these streets. Provision was made for carrying the tracks in an open subway, with the necessary retaining walls and underpinning of all structures along the line, from a point near Broad St. to Hamilton St., by a tunnel from Hamilton St. to a point near Taney St., and thence by an open subway to a point near Thirtieth St. An entire reconstruction of the sewerage system between Twelfth and Thirtieth Sts. was made necessary, as well as the alteration and reconstruction of gas and water mains, electrical conduits and other municipal structures.

Twelfth, Thirteenth, Broad and a few of the other intersecting streets required some change of grade.

The lengths of the various classes of work to be constructed are as follows:

Elevated structure	959 ft.
Open subway	6 330 "
Tunnel	2 711 "
Total	10 000 ft.

On August 31st, 1894, a formal agreement* was entered into between the Receivers of the Philadelphia and Reading Railroad and the city,

* The form of agreement and ordinance in full have been filed in the Library of the Society for reference.

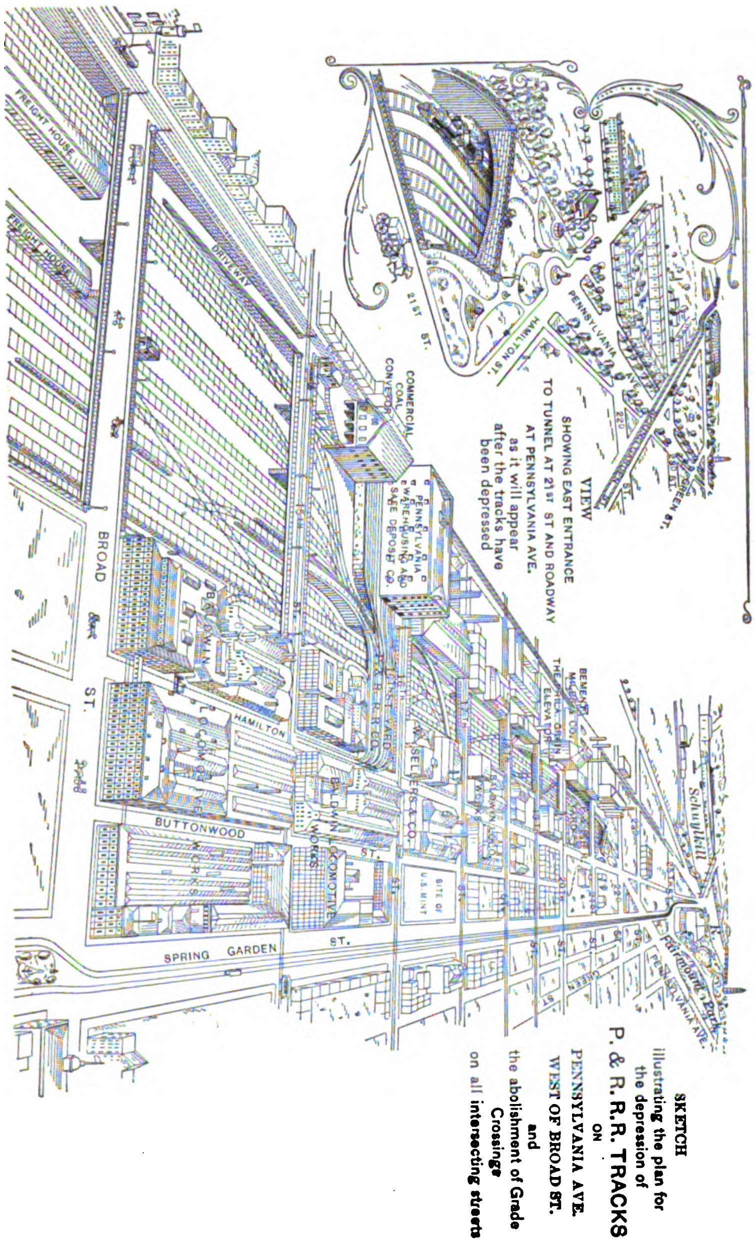

SKETCH
illustrating the plan for
the depression of
P. & R. R.R. TRACKS
ON
PENNSYLVANIA AVE.
WEST OF BROAD ST.
and
the abolishment of Grade
Crossings
on all intersecting streets
VIEW
SHOWING EAST ENTRANCE
TO TUNNEL AT 21ST ST AND ROADWAY
AT PENNSYLVANIA AVE.
as it will appear
after the tracks have
been depressed
FREIGHT HOUSE
FREIGHT HOUSE
DRIVEWAY
BROAD ST.
HAMILTON
BUTTONWOOD
SPRING GARDEN ST.
BALDWIN LOCOMOTIVE WORKS
COMMERCIAL COAL CONVEYOR
PENNSYLVANIA WAREHOUSING AND SAFE DEPOSIT CO.
SITE OF U.S. MINT
WM. SELLERS & CO.
HAMILTON ST.
PENNSYLVANIA AVE.
21ST ST.
GREEN ST.
Schuylkill
Fairmount Park
PENNSYLVANIA AVE.

FIG. 2.

in compliance with the terms of the ordinance, which provided that before the ordinance should go into effect the Railroad Company should file its acceptance of the provisions contained and a covenant should be entered into, in form to be approved by the City Solicitor, to perform all the provisions of the ordinance as far as they relate to the Railroad Company.

On the same day they also filed a bond for $500 000, as required, as security for their payment to the City of half the total cost of the work.

On August 31st, 1894, the general plans for the construction of the work were approved, as required by the ordinance, and on August 30th bids were received for the work upon the entire sewerage system, the detailed plans for which had been prepared in anticipation.

The ordinance specifically provides that all the engineering work is to be done by the City under the supervision of the Director of the Department of Public Works, and that the plans and specifications, after being prepared, are to be submitted to the officers of the Railroad Company for their approval, and copies filed with the Railroad Company and the City.

Provision is also made for the necessary revisions of the lines and grades of the streets in order to carry out the work.

Fig. 3 is a general plan of the work and the surroundings.

Soundings.

A large number of soundings, by the water-jet process, were made along the line of the subway and on the route of the sewers, in order to give the contractors for the sewers an idea of the character of the material to be excavated, and to aid in making intelligent designs for the work of underpinning the buildings and for the main work of construction of the subway proper. Subsequent excavations, adjacent to these soundings, developed the fact that they had been made carefully, and were therefore of the greatest value in designing the work and making intelligent estimates. On account of the soft nature of the rock in many places, shown on the soundings as hard rock, it would have been well if a limited number of diamond drill borings had been made in selected localities along the line, in order to determine the nature of the rock more exactly.

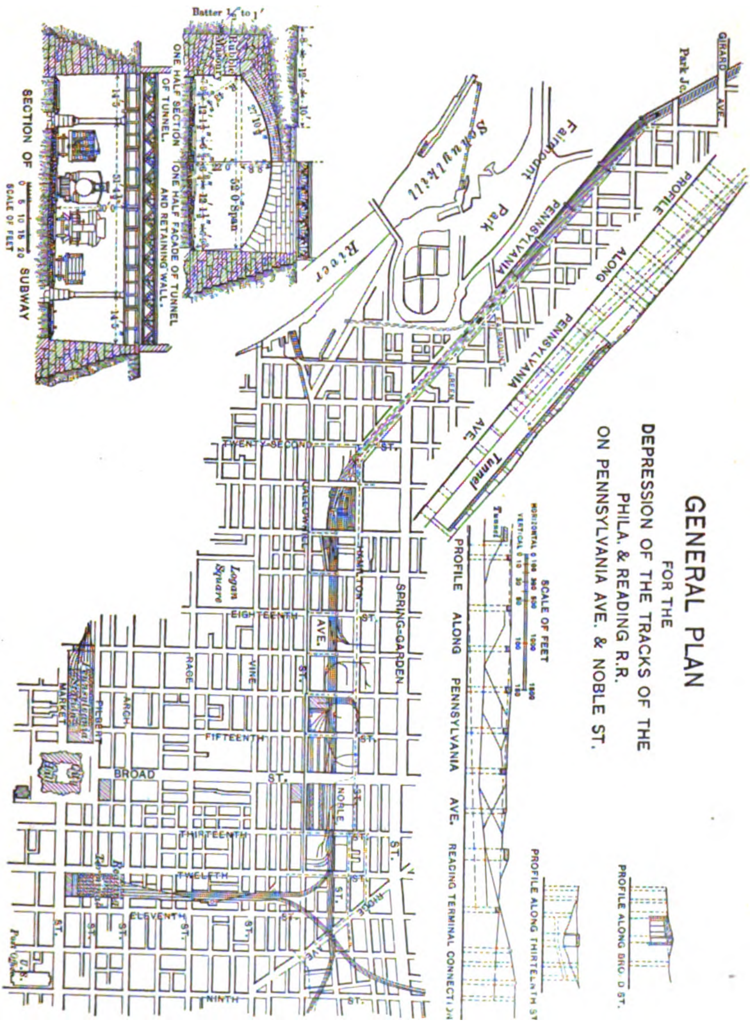

GENERAL PLAN
FOR THE
DEPRESSION OF THE TRACKS OF THE
PHILA. & READING R.R.
ON PENNSYLVANIA AVE. & NOBLE ST.
SCALE OF FEET
PROFILE ALONG PENNSYLVANIA AVE.
PROFILE ALONG PENNSYLVANIA AVE.
PROFILE ALONG THIRTEENTH ST.
PROFILE ALONG BROAD ST.
READING TERMINAL CONNECTION
Tunnel
Schuylkill River
Fairmount Park
Park Jc.
GIRARD AVE.
TWENTY-SECOND ST.
CALLOWHILL
HAMILTON ST.
SPRING GARDEN
Logan Square
EIGHTEENTH ST.
FIFTEENTH ST.
BROAD ST.
NOBLE ST.
THIRTEENTH ST.
TWELFTH ST.
ELEVENTH ST.
NINTH ST.
RACE
VINE
ARCH
MARKET
FILBERT
RIDGE AVE.
Batter 1/2" to 1'
Rubble Masonry
32' 0" Span
ONE HALF SECTION OF TUNNEL.
ONE HALF FACADE OF TUNNEL AND RETAINING WALL.
SECTION OF SUBWAY
SCALE OF FEET

Fig. 3.

Sewers.

In order to drain the subway, and to provide for the old sewers cut by the excavations, three new and independent systems of main sewers were necessary to accomplish this purpose in the most satisfactory and economical manner, there being no existing systems at a sufficiently low level.

The three systems into which the sewers for carrying on this work were divided are as follows:

First. The Twenty-fourth Street and Pennsylvania Avenue System.—This system provides for the drainage west of Twenty-fourth St. on the north side of Pennsylvania Ave., intercepting all the cross sewers between Twenty-fourth and Thirtieth Sts., carrying the same beneath the tunnel at Twenty-fourth St., down Twenty-fourth St. to Powelton Ave., and on the line of Powelton Ave. to the Schuylkill River. This system was divided, for construction purposes, into two contracts.

Second. Callowhill Street System.—This system provides for the drainage of the subway and tunnel from Thirteenth to Twenty-third Sts. by means of a main sewer on Callowhill St. (which runs parallel to Pennsylvania Ave., east of Twentieth St.) with appurtenant sewers on the cross streets leading under the subway and receiving through well-holes the sewage from the old high-level sewers to the north. From Twenty-third St., the main sewer follows Powelton Ave. to Twenty-fourth St., passes along Twenty-fourth St. to Wood St. and thence to the Schuylkill River. This system is divided, for construction purposes, into three contracts.

Third. Twelfth Street System.—This system is designed to intercept the old Thirteenth St. main sewer, carry it down Buttonwood St. to Twelfth St., thence along Twelfth St. to Carlton St., where it discharges into an old main sewer on the Delaware River water-shed. This system is in one contract.

Detail plans were prepared for these sewers and special specifications written upon the same lines as those in use by the Sewer Division of the Bureau of Surveys (the City Bureau having charge of all sewer work). The sewers were contracted for at a price per lineal foot for each size of completed sewer, including excavation, refilling and all appurtenances shown on the drawings, such as manholes, wellholes, spurs, etc.

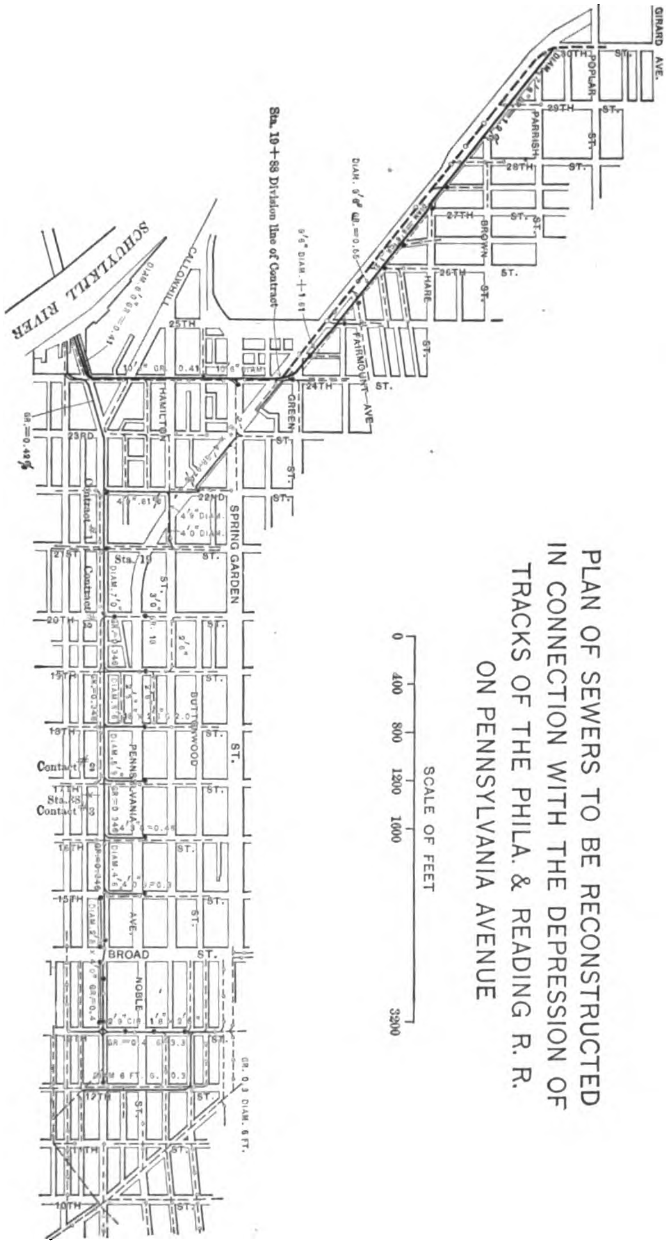

PLAN OF SEWERS TO BE RECONSTRUCTED
IN CONNECTION WITH THE DEPRESSION OF
TRACKS OF THE PHILA. & READING R. R.
ON PENNSYLVANIA AVENUE
SCALE OF FEET
0
400
800
1200
1600
3200
Sta. 19+88 Division line of Contract
SCHUYLKILL RIVER
GIRARD AVE.
POPLAR
PARRISH
BROWN
FAIRMOUNT AVE.
CALLOWHILL
HAMILTON
GREEN
SPRING GARDEN
BUTTONWOOD
PENNSYLVANIA AVE.
BROAD ST.
NOBLE
Contract
Sta. 19
Sta. 38
25TH
24TH
23RD
22ND
21ST
20TH
19TH
18TH
17TH
16TH
15TH
12TH
11TH
10TH

Fig. 4.

These specifications,* while covering the whole work of construction in detail, specify some features which are rather different from usual practice, and are, therefore, worthy of special attention.

It was directed that all sewers, wherever practicable, were to be constructed in tunnel, the work being carried on day and night.

All appliances for removing excavated material were to be specially adapted for the prevention of interference with travel, and were to be subject to the approval of the chief engineer.

When constructed in tunnel, the space between the outside of the sewer and the roof and sides of the tunnel was to be filled compactly with rubble masonry or concrete, as directed.

On account of the light grades, the inverts on curves were to be plastered with Portland cement mortar, ½-in. thick, to reduce the friction.

In the general clauses relating to the responsibility of the contractors and to maintenance, provision is made to settle disputes between the contractors on the several sections as to the disposition of drainage by making the Director of the Department of Public Works the arbiter. The maintenance of all new work is to be cared for by the contractor or his sureties for five years from its completion. The manner, the times of carrying on the work and the force used in construction are to be subject to the approval of the Chief Engineer.

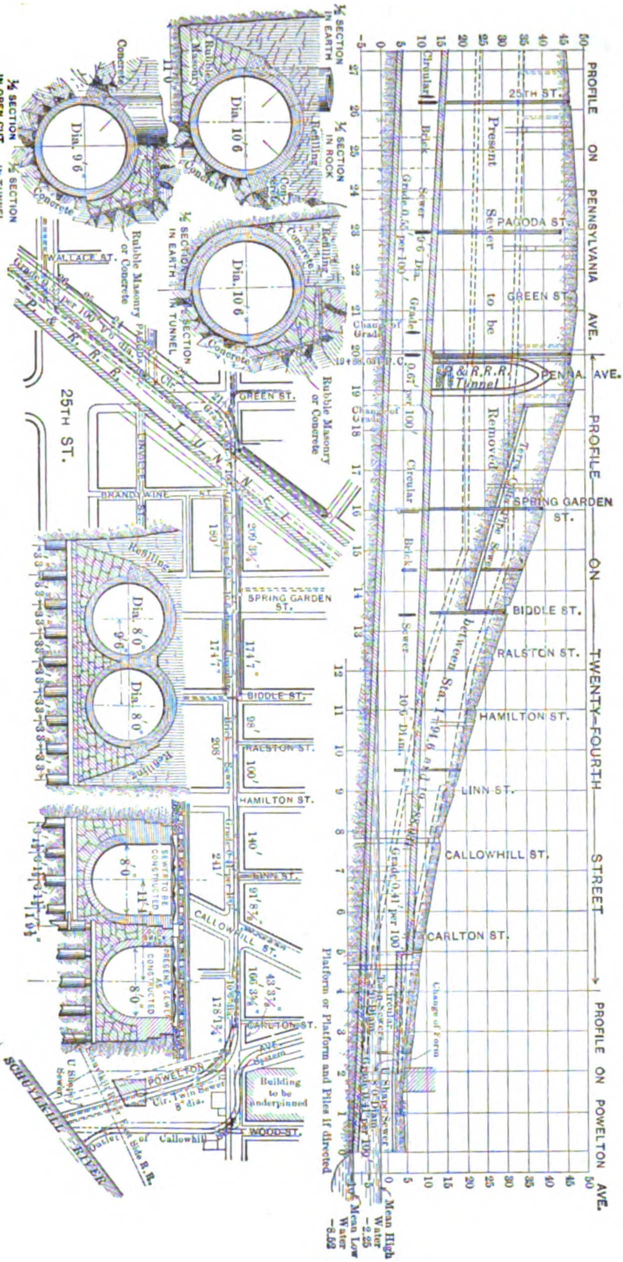
The general drainage plan of this work is shown in Fig. 4.

The time for the completion of the work was fixed by the department at four months from the date of the order to proceed.

Bids were received on August 30th, 1894, and on September 4th and 5th contracts were executed, as follows:

	Contractors.	Limit of Contract.
Twenty-fourth St. and Pennsylvania Ave. System:		
Contract No. 1.	Ryan & Kelly	$102 000
" No. 2.	J. A. Mundy & Bro	138 000
Callowhill St. System:		
Contract No. 1.	C. P. Grim & Co	94 000
" No. 2.	Geo. W. Ruch	75 000
" No. 3.	John McCann	47 000
Twelfth St. System:		
	John McCann	25 000

* The full specification has been placed on file in the Library of the Society.



MAIN SEWER
ON
24th ST. AND PENNSLVANIA AVE.
Between the outlet at Powelton Ave., and 30th St.
FIG. 5.

The order to proceed with the work was given to all contractors alike on September 10th, 1894, and it was directed that the work be proceeded with, day and night.

Detailed descriptions of the work on each contract follow.

Twenty-fourth Street and Pennsylvania Avenue System. Contract No. 1.—Located on the line of Powelton Ave., Twenty-fourth St. and Pennsylvania Ave., from the Schuylkill River to Station 19 + 88. Ryan and Kelley, Contractors.

This contract extended from the Schuylkill River at the foot of Powelton Ave. to the north of and alongside of an old 8-ft. U-shaped sewer; passed, with very close clearance, under the tracks of the Schuylkill River East Side Railroad (Baltimore and Ohio System) and, at a distance of about 205 ft. from the outlet, changed to an 8-ft. twin sewer. At the wharf, the elevation of the inside bottom of the invert is — 7.5, Philadelphia City datum. This datum is 2.25 ft. above mean high water and 8.52 ft. above mean low water in the Delaware River at Philadelphia. It is thus seen that the inside bottom of the sewer is only 1.02 ft. above mean low water. From this point, for a distance of about 150 ft., the sewer was constructed upon piles with a yellow pine platform and masonry cradle. The details of this section of the sewer are shown in Fig. 4. At about Station 1 + 50 the sewer runs beneath an old warehouse. Here, the piles were omitted and a timber platform 22 ins. thick was substituted. As will be seen by reference to Fig. 5, the pile work was below low tide, and the excavation for a distance of over 100 ft. from the outlet was through an old wharf, making the use of a coffer dam impracticable. A diver was employed to cut off the tops of the piles after they had been driven, and the platform and cradle were placed in position at low tide, bulkheads being used wherever practicable to facilitate the work.

The U-shaped sewer was covered with steel I-beams and buckle-plates filled in on top with concrete. This construction was necessitated on account of the low level of the railroad tracks crossing at this point. It will be noticed by reference to the plans that the new main sewer on Twenty-fourth St. is 10 ft. 6 ins. in diameter. This could not be carried under the railroad tracks and preserve the required elevation of the inside bottom; hence, a separating chamber, placed at the intersection of Twenty-fourth St. and Powelton Ave., and leading to a pair of circular sewers 8 ft. in diameter, was designed. Where these

twin sewers passed under the tracks the section was again changed, as noted above. The U-shaped portion of the old sewer on this line was retained from the old warehouse to the outlet, and the new sewer was built to the north of it.

From the point where the twin sewers commence, to the end of the separating chamber, the foundation was upon two layers of 3-in. yellow pine plank laid at an angle of 60° to each other. From the separating chamber to the north side of Callowhill St. a simple masonry cradle was used, and at about this point the excavation was largely through rock. Just south of Hamilton St. tunneling was started, and continued to the end of the contract at Station 19 + 88. It was entirely through rock, with the exception of a gravel pocket beneath Pennsylvania Ave. A tunnel was also driven beneath Callowhill St. through soft material so as not to interfere with traffic on the street.

The old sewer on Twenty-fourth St. was 10 ft. in diameter, and was very old. There was a heavy and constant flow of sewage through it, and, owing to the heavy grade, the invert in many places was practically worn out and washed away. In several places, and for considerable distances, there was no brick work whatever remaining. The new sewer was constructed on essentially the same alignment as the old one, but with a much flatter grade. Up to a point near Hamilton St., it was necessary to carry the water during construction by means of a flume of wood, rectangular in section and 6 ft. square, constructed under the west sidewalk of Twenty-fourth St., and to remove the old sewer entirely, in order to build the new one. This flume extended from Hamilton St. to about Station 1 + 90 where it discharged into the old U-shaped sewer. From Hamilton St. north, by careful working, a tunnel was driven, and the new sewer was constructed beneath the old one, which was in active service. Several breaks occurred in the bottom of the old sewer, allowing the sewage to fall into the workings beneath, but in no case causing injury or loss of life. Entrance to the tunnel was obtained through four side-shafts with drifts into the main tunnel. These shafts were generally located in the small cross streets off the line of Twenty-fourth St.

Of the total length of main sewer constructed, 1 988.03 ft., the length in tunnel was 1 001.03 ft. Ingersoll-Sergeant drills with 3½-in. cylinders were used, fed with compressed air through 3-in. and 4-in. service pipe from an Ingersoll-Sergeant Class "A" compressor with a

capacity of twelve drills. The compressors were supplied with steam from two 80 H.-P. horizontal boilers. Ten hoisting engines and twelve pumps constituted part of the contractors' plant.

The blasting for the flume, referred to, seriously damaged a number of houses on the west side of Twenty-fourth St., which were in a very bad condition previous to beginning work. This damage was made good by the contractor, under the terms of his contract.

Progress in making the excavation for the new sewer tunnel was

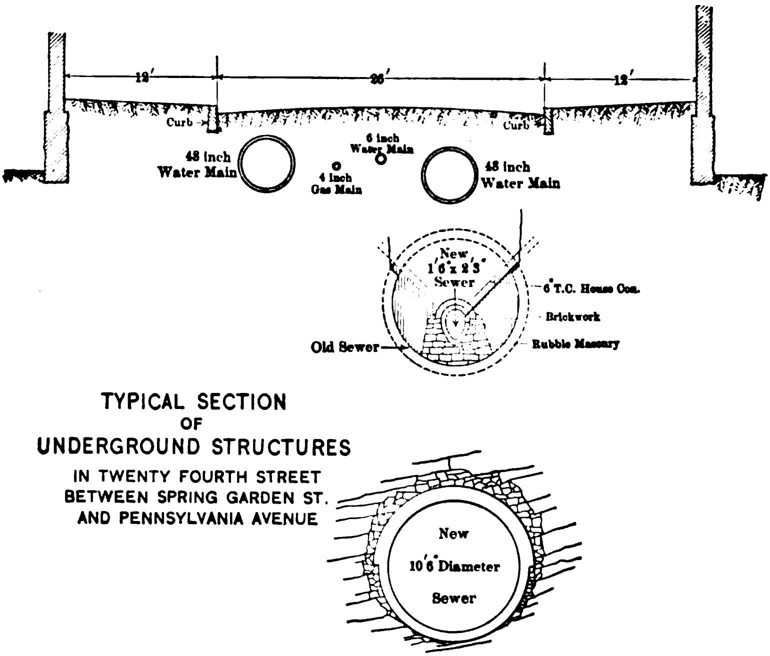


FIG. 6.

necessarily slow, on account of the dangerous proximity of buildings, the old sewer and one 6-in. and two 48-in. water mains in the street above the tunnel. Fig. 6 is a section through the street, between Spring Garden St. and Pennsylvania Ave., in which these structures were found. Work was begun September 10th, 1894, and was pushed vigorously day and night. The last sewer connection was made May 6th, 1895. The photographs on Plate XVIII were taken from the open cut and tunnel work on this contract.

FIG. 1.—OPEN CUT ON 24TH STREET, NORTH OF CALLOWHILL STREET.

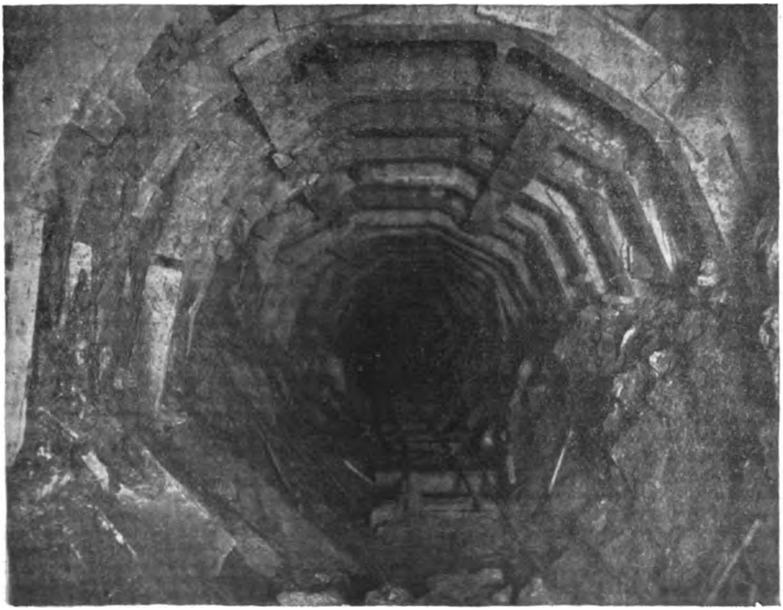


FIG. 2.—TUNNEL ON 24TH STREET, UNDER PENNSYLVANIA AVENUE.

TABLE No. 1.—LENGTHS, SIZES AND PRICES PAID.

Sizes and Shapes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
8 ft. diameter, U-shaped............	$47.78	205.30	....
8 ft. diameter, twin.................	58.51	293.32	....
10 ft. 6 ins. diameter with 12-in. pipe.	43.00	1 337.08	877.70
9 ft. 6 ins. diameter..................	45.00	123.33	123.33
Separating chamber................	86.02	29.00	....
		1 988.03	1 001.03

Total payment to contractors: $92 586.42.

Twenty-fourth Street and Pennsylvania Avenue System. Contract No. 2.—Located on the line of Pennsylvania Ave. from Station 19 + 88 to Thirtieth St. James A. Mundy and Brother, Contractors.

This is a continuation of Contract No. 1, and completes the system by making connection with the old sewer on Thirtieth St. The sewer, where it crosses under Twenty-fourth St., is 9 ft. 6 ins. in diameter and gradually decreases in size to 7 ft. 6 ins., which is the size of the old sewer on Thirtieth St. Of the total length of main sewer on this contract, 3 470.66 ft., the length in tunnel was 2 878.02 ft. Access to the tunnel was obtained through sixteen shafts, generally located where manholes or wellholes occurred. Very little soft material was encountered in making the excavation, although frequent timbering in isolated places was required, on account of the dip and seamy character of the rock, which was a mica schist of very irregular character and hardness, in some places being badly decomposed, even where 40 ft. beneath the natural surface. At the lower end of the contract, on the curve running west from Twenty-fourth St., the rock was unusually slippery when exposed to the air, and required careful watching. Just west of Twenty-eight St., the tunnel excavation passed through a fill, formerly the bed of an old creek, and considerable difficulty was experienced in making the timbering stable.

From Brown St. to Thirtieth St. the sewer was built in open cut.

From Twenty-fourth St. to Fairmount Ave. the tunnel passed beneath a large number of dwelling houses and an ice-manufacturing plant, the latter in active operation and the former occupied. These structures were finally removed, upon the widening of Pennsylvania Ave., and the sewer is now in the bed of the street. With the exception of breaking a 48-in. water main, no serious injury was done

by the blasting. From Fairmount Ave. westward, the line of the sewer is generally distant from any buildings.

Drills of the Rand type, with 3¼-in. cylinders, were used, and were fed with compressed air through 4, 6 and 8-in. service pipes from two Rand compressors with a joint capacity of thirteen drills. The compressors were supplied with steam from two 75 H.-P. horizontal boilers. Seven hoisting engines and six pumps constituted part of the contractors' plant.

Work was begun September 10th, 1894, and was pushed day and night. The last sewer connection was made April 16th, 1895. Fig. 1, Plate XIX, shows the work in tunnel on this contract.

TABLE No. 2.—Lengths, Sizes and Prices Paid.

Sizes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
9 ft. 6 ins. diameter............	$49.33	1 382.70	1 382.70
8 ft. 6 ins. diameter............	35.99	870.26	870.26
7 ft. 6 ins. diameter............	30.38	1 217.70	625.06
		3 470.66	2 878.02

Total payment to contractors: $126 496.15.

Callowhill Street System. Contract No. 1.—Located on the line of Wood St., Twenty-fourth St., Powelton Ave. and Callowhill St. from the Schuylkill River to Station 19 at Twenty-first St., with appurtenant sewers on Twenty-second St., Hamilton St. and Pennsylvania Ave., C. P. Grim and Company, Contractors.

This contract is the outlet and most difficult section of the Callowhill St. System. From a point near Twenty-fourth St. to the Schuylkill River, the sewer followed the line and approximately the grade of the old Wood St. sewer, which is about 4 ft. in diameter. Near the crossing of the Baltimore and Ohio Railroad, the new sewer cut through part of the old wharf referred to on the Twenty-fourth St. System. At the outlet, the elevation of the inside bottom of the invert is — 8.17, or only 0.35 ft. above mean low water.

From the outlet to Station 1 + 40 the sewer is 8 ft. 6 ins. in diameter with an 18-in. arch, and is constructed upon a platform placed upon piles and with a masonry cradle. The piles and platform extended to about Station 1 + 40, where a timber platform was

FIG. 1.—TUNNEL ON PENNSYLVANIA AVENUE, WEST OF 26TH STREET.

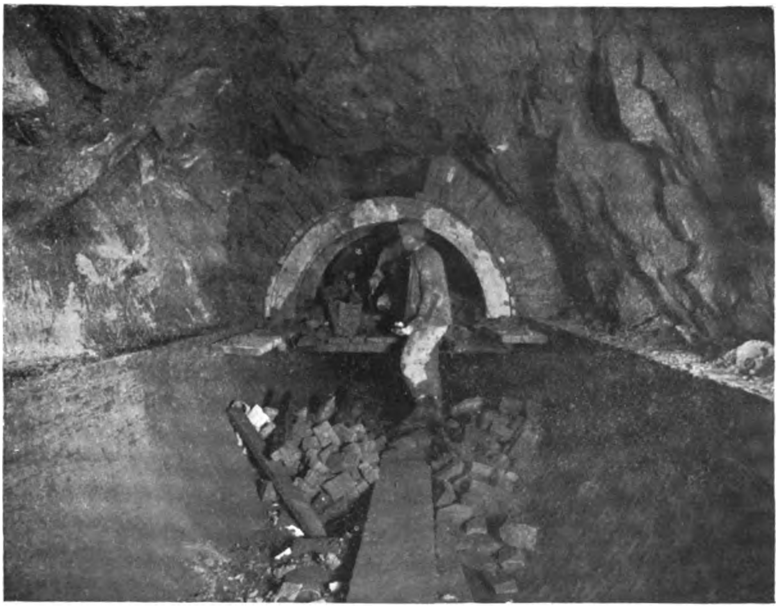


FIG. 2.—SEWER IN TUNNEL ON CALLOWHILL STREET.

commenced. During the construction of the new sewer on the line of Wood St. it was necessary to flume the sewage from the old sewer, and, at the outlet, to work only at the lowest stages of the tide. It was impossible to place bulkheads at any point to prevent ingress of the tide, on account of the structure of the old wharf.

At the southeast corner of Twenty-fourth St. and Powelton Ave. the sewer approaches very closely to the three-story brick building of Bement, Miles and Company, used as a store house for patterns of machine tools, and, on account of the admittedly bad character of the foundation under the walls of this building, it was not considered advisable to proceed with the excavation without first underpinning the building. A supplementary contract was therefore made with the contractors for this underpinning.

The special specifications prepared for this work required the excavation to be made by sinking shafts, not adjacent to each other, and shoring the sides carefully where necessary. All loose or imperfect stones in the bottom of the old foundation were required to be removed, until such portions of the same were reached as were compact and the stones of large size. At the center of each section (generally 4 ft. long), and under the prepared bottom of the foundation wall, a timber of long-leaf yellow pine 10 by 10 ins. in section was to be placed and firmly wedged up with iron or steel wedges. The space around the timber and under the wall was filled with masonry of stone with specially flat beds (the stone from Conshohocken, Montgomery County, Pa., was specified), laid in Portland cement mortar, mixed in the proportion of 1 part cement to 1 part sand. This masonry was required to be wedged up firmly so that no space was left for settlement.

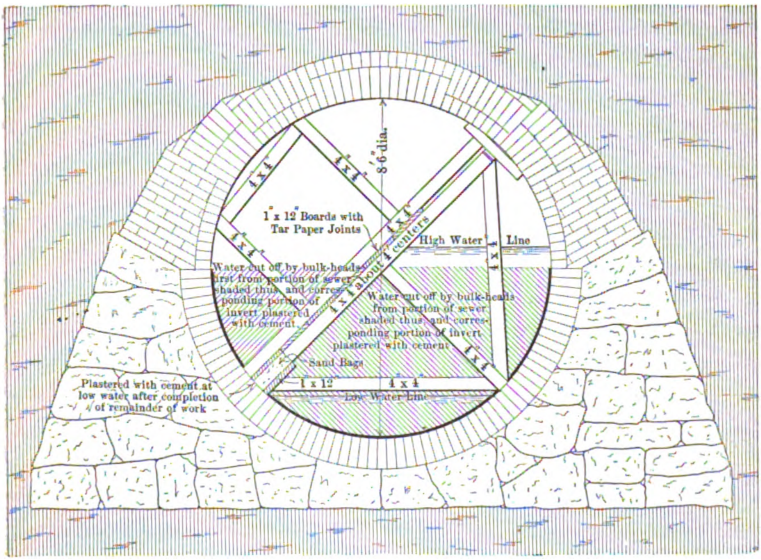
The excavation was carried down through river mud and sand for a distance of 20 ft., and was stopped on a bed of compact gravel below the bottom of the excavation required for the sewer. A bed of concrete about 4 ft. wide and 3 ft. deep was laid as a footing course. The building thus underpinned contained a number of cracks previous to the operation, but no new cracks were started, nor were any of the old ones increased in size. The price paid for the underpinning, including the excavation and all appurtenances, was $18 per cubic yard.

On the line of Powelton Ave., the excavation was in open cut through a bed of gravel as far as Callowhill St., where a tunnel was

driven beneath this street and Twenty-third St. in order to keep the streets open to travel. While driving this tunnel, and on the line of Twenty-third St., an old sewer, about 3 ft. in diameter, was encountered, of which there was no record on the city drainage plans. This sewer carried very large volumes of water at certain hours of the day when the dye mills in the vicinity emptied their tanks. This caused the contractors much annoyance and expense in maintaining the flow of the sewage. From Twenty-third St. to a point west of Twenty-second St., the excavation was in open cut, the contractors using, on this section, a Carson sewer-trench machine. A second unrecorded sewer, running diagonally beneath the adjacent houses, was cut about the middle of this block. The volume of water carried by it, however, was very small and was easily cared for. From a point just west of Twenty-second St. to Station 19, the end of the contract on Callowhill St., and to the end of the appurtenant sewers at Twenty-first and Hamilton Sts. and Twenty-second St. and Pennsylvania Ave., the excavation was made in tunnel, with the exception of a short length at Twenty-first and Hamilton Sts. The tunnel was entered by means of eight shafts so located as to be off the main streets. The total length of the sewers on this contract was 3 094.61 ft., of which the length constructed in tunnel was 1 751.87 ft. Under Callowhill St. the tunnel was in soft ground, the remainder was through rock of varying degrees of hardness. Drills of the Ingersoll-Sergeant, Rand and Kerner types, with 3-in. cylinders, were used. They were fed with compressed air through a 4-in. service pipe from a Blake duplex compressor with a capacity of eight drills. The compressor was supplied with steam from two vertical boilers with a combined horse-power of 70. The contractors had in use four hoisting engines and ten pumps.

The contractors on this section did not prosecute their work as rapidly as they should, and, as they finished their brick work, they failed to plaster the inverts on curves as required by the specifications. As a consequence, they were compelled to care for the full flow of the sewers on the contracts above them, and plaster the inverts and maintain the flow of the sewer at the same time. In order to accomplish this, a longitudinal flume, the full length of the curve, was constructed, as shown in section in Fig. 7. Bulkheads of sand bags were then built at each end of the flume in the spaces shown by the smallest shaded

area, and the flow of sewage or the returning tide made to pass beneath. The smaller segment of the invert, being free from water, was then plastered. As soon as the plaster had set, the bulkheads were removed and placed in the spaces shown by the largest shaded area, and the flow made to pass on top of the flume. The larger segment was then plastered. After the flume was removed from the sewer, the spaces on the invert occupied by the framing were plastered at low water, they being purposely arranged so as to be above water at low tide and at ordinary stages of the flow in the sewer. During its



SECTION OF CALLOWHILL ST. SEWER
SHOWING FLUME USED FOR PLASTERING INVERT ON CURVES
SCALE OF FEET
0 1 2 3 4 5 6

Fig. 7.

use the flume was endangered by two severe rain storms. Sufficient warning was given, however, to enable men to enter the sewer and remove the bulkheads by throwing the sand bags on the bottom, thus reducing the obstruction to a minimum. No damage resulted in either case.

The plastering upon brick over which sewage had been running was a piece of work which required great care and skill, in order to make a good finish. In some cases, where the ground-water was

large in amount, weep holes were made through the brick. At such places, the bricks were thoroughly soaked with water, and there was considerable trouble in making the plaster adhere. In some cases, the grease from the sewage interfered with the proper adhesion, and had to be burnt off with plumber's blow-pipes. Experience with the conditions soon resulted in a very satisfactory piece of work. A quick-setting neat cement, worked as dry as possible, being used where the amount of water in the brick was greatest.

Work was begun September 10th, 1894, was pushed vigorously for a few months, and then carried on in a dilatory manner. The last sewer connection was made in November, 1895, and the plastering of the inverts was finished February 26th, 1896.

TABLE No. 3.—LENGTHS, SIZES AND PRICES PAID.

Sizes and shapes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
8 ft. 6 ins. diameter, 18-in. arch	$72.50	140.00	....
" " " 13½-in. arch	26.75	1 244.24	196.00
7 ft. diameter	27.15	516.73	516.73
4 ft. 9 ins. diameter	20.00	692.64	692.64
4 ft. diameter	17.25	250.00	95.50
3 ft. diameter	20.50	251.00	251.00
4 ft. by 2 ft. 8 ins., egg-shaped	8.00	not built.	....
		3 094.61	1 751.87

Total payment to contractors: $80 342.75.

The 4 ft. by 2 ft. 8 in. egg-shaped sewer was not constructed, on account of its close proximity to the north wall of the tunnel, which had to be excavated in rock immediately adjacent. It was omitted, therefore, in this contract, and constructed after the excavation of the tunnel wall was made.

Callowhill Street System. Contract No. 2.—Located on the line of Callowhill St., from Station 19 at Twenty-first St., to Station 38 at Seventeenth St., with appurtenant sewers on Seventeenth, Eighteenth, Nineteenth and Twentieth Sts., George W. Ruch, Contractor. This was a continuation of Contract No. 1, and was the middle contract of the system.

The whole of the 2 953.20 ft. of sewers constructed on this contract was built in tunnel, access to the headings being obtained through twelve shafts. At the western end of the line, the tunnel was driven through rock, and very little or no timbering was required. From

PLATE XX.
PAPERS AM. SOC. C. E.
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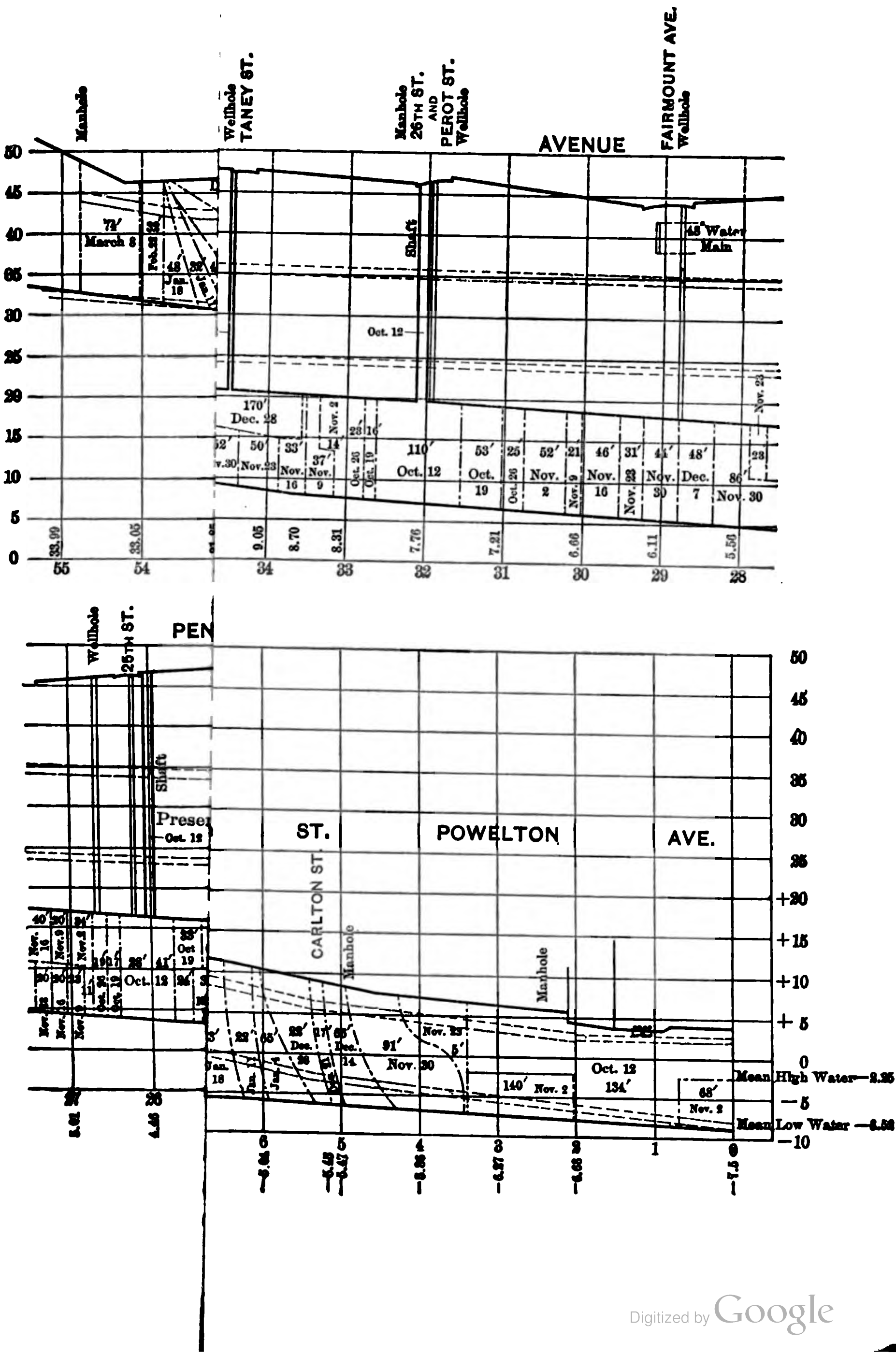
WEBSTER AND WAGNER ON SUBWAY SEWER CONSTRUCTION.



Station 19 to a point near Nineteenth St., the excavation was through rock which decreased in hardness as the work progressed toward the east. Rock was encountered on Twentieth St., that under Pennsylvania Ave. being specially hard. On the other hand, the sewer on Seventeenth St. was driven through a micaceous sand, very slippery and dangerous to handle, and requiring close sheathing in the headings. The most serious accident on the work occurred on the Callowhill St. sewer, where it passed under the old high-level sewer on Nineteenth St. A cave-in occurred at this point, caused by insufficient timbering, the sewage from above filling the entire workings below. There was also trouble on Callowhill St., between Twentieth and Twenty-first Sts., caused by blasting. The blasts were so severe as to affect the adjustment of machine tools in the shops of Messrs. Bement, Miles and Company, immediately adjoining the line of the sewer, and spoiling some work which was on the machines. After several cautionary orders, the Department was compelled to direct the detailed operations of firing the charges, and, as a result, the contractor, at the termination of the work, sued the city for damages resulting from the issuance of such an order. The suit was entered by the contractor in the sum of \$57 377.46, afterward revised, as agreed upon by counsel, to \$52 785.16. The suit was tried in November, 1897, and was interesting on account of the testimony of an expert nature, presented by the contractor to show that the chief engineer had exceeded his authority by issuing orders directing the manner in which the blasting was to be done, when no special method was specified. The court ruled that the chief engineer had the right to issue such instructions if they were reasonable.

Testimony was presented by the city to show that the progress made in driving the tunnels was as great after issuing the order as it had been before. As this testimony could not be refuted, the case was settled, and a verdict of \$8 705.13 was awarded the contractor. This verdict meant a practical victory for the city, as it represented retained moneys and interest on the same held by the city, pending the settlement of the case.

Drills of the Ingersoll-Sergeant type with 3½-in. cylinders were used. The compressed air was supplied through 3, 4 and 5-in. service pipes from a Norwalk single compressor, with a capacity of twelve drills. The compressor was supplied with steam from two

65 H.-P. vertical boilers. In addition to this machinery the contractor had in use five hoisting engines and five pumps.

Work was begun on September 20th, 1894, and the last sewer connection was made June 3d, 1895. Fig. 2, Plate XIX, shows the sewer construction in tunnel on Callowhill St., between Twentieth and Twenty-first Sts.

TABLE No. 4.—Lengths, Sizes and Prices Paid.

Sizes and shapes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
7 ft. diameter	$29.65	967.00	967.00
6 ft. 6 ins. diameter	29.54	478.00	478.00
5 ft. 9 ins. diameter	26.29	455.00	455.00
3 ft. diameter	16.27	257.45	257.45
2 ft. 9 ins. diameter	16.27	265.25	265.25
3 ft. 3 ins. by 2 ft. 2 ins., egg-shaped	9.77	265.25	265.25
2 ft. 6 ins. by 1 ft. 8 ins., egg-shaped	10.35	265.25	265.25
		2 953.20	2 953.20

Total payment to contractors: $69 662.71.

Callowhill Street System. Contract No. 3.—Located on the line of Callowhill St., from Station 38 at Seventeenth St., to Thirteenth St., and on Thirteenth St. from Carlton St. to Buttonwood St., with appurtenant sewers on Fifteenth and Sixteenth Sts. John McCann, Contractor.

The total length of sewers on this contract was 2 467.97 ft., of which the length constructed in tunnel was 2 060.97 ft. Access was obtained through nine shafts. A considerable portion of the sewer on Sixteenth St., and the main sewer on Callowhill St. from east of Sixteenth St. to Station 38, was in rock; the remainder of the work was through soft material, much of it being a slippery micaceous sand, comparatively free from water. While the department would have allowed open cutting east of Broad St., on account of the soft character of the material, and because the depth of the sewer was not very great, the contractor elected to built the sewer in tunnel, and made very satisfactory progress, and without accident.

On account of the grade of Thirteenth St. being changed materially by the plans of the subway, it was deemed inadvisable by the department to dig up the street and construct the sewer at the time the rest of the sewer work was being done, on account of the double incon-

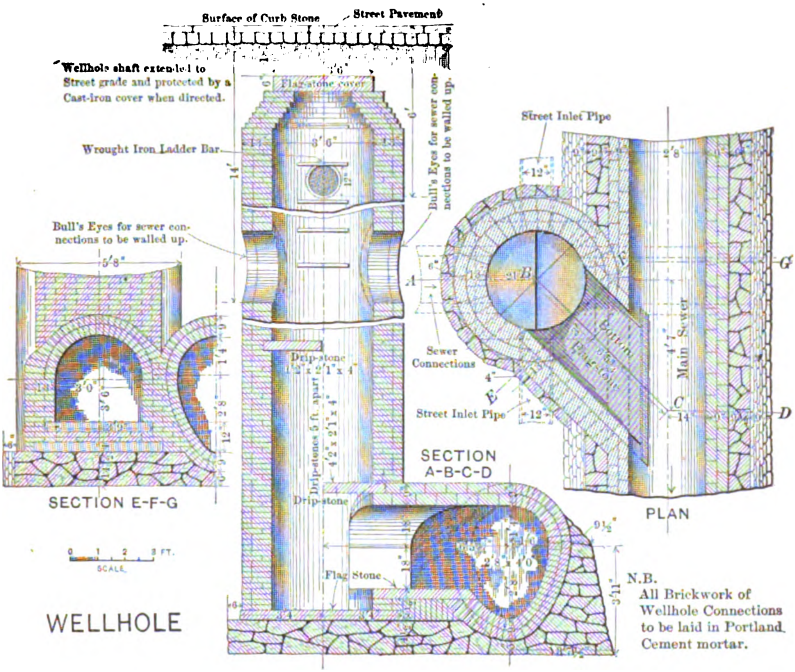
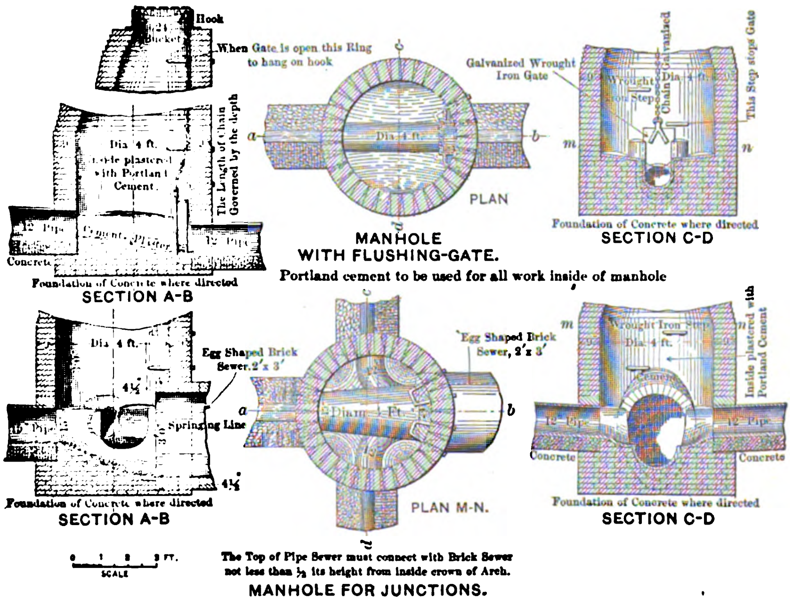


Fig. 8.



GENERAL DETAILS FOR PIPE SEWERS.

Fig. 9.

venience to the public. No work was done on Thirteenth St., therefore, under this contract.

The contractor had no power drills or compressors on the work. All the drilling was done by hand.

Work was begun September 10th, 1894, and the last sewer connection was made May 18th, 1895. This connection was delayed, on account of the failure of the contractor on Contract No. 2 to complete his tunnel at the upper end of his section.

TABLE No. 5.—LENGTHS, SIZES AND PRICES PAID.

Sizes and Shapes.	Contract price, per lineal foot.	Total length, in feet.	Length in tunnel, in feet.
5 ft. 9 ins. diameter	$27.90	390.52	390.52
4 ft. 6 ins. diameter	12.84	488.21	488.21
4 ft. by 2 ft. 8 ins., egg-shaped	10.50	1 056.92	651.92
2 ft. 9 ins. diameter	11.00	not built.	not built.
2 ft. 6 ins. diameter	7.75	38.50	38.50
2 ft. 6 ins. by 1 ft. 8 ins., egg-shaped	5.75	not built,	not built.
12-in. T. C. pipe	2.40	" "	" "
4 ft. 3 ins. diameter	11.00	273.72	273.72
4 ft. diameter	11.70	253.75	258.75
Junction chamber	104.50	14.35	14.35
		2 467.97	2 060.97

Total payment to contractor: $35 976.81.

Twelfth Street System.—This contract extends on Twelfth St., Buttonwood St. and Thirteenth St., from Carlton St. to Whitehall St. John McCann, Contractor.

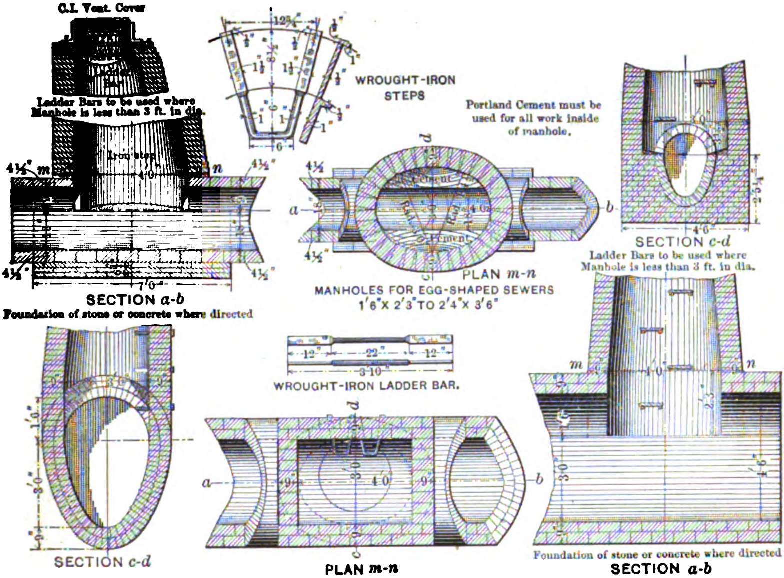
The sewer on this system was near the surface of the street and was constructed entirely in open cut. There were no points of special interest in connection with it.

Work was started September 10th, 1894, and the connection to the old sewer was made on December 29th, 1894.

TABLE No. 6.—LENGTH, SIZE AND PRICE PAID.

Size.	Contract price, per lineal foot.	Length.
6 ft. diameter.	$16.00	1 398.00

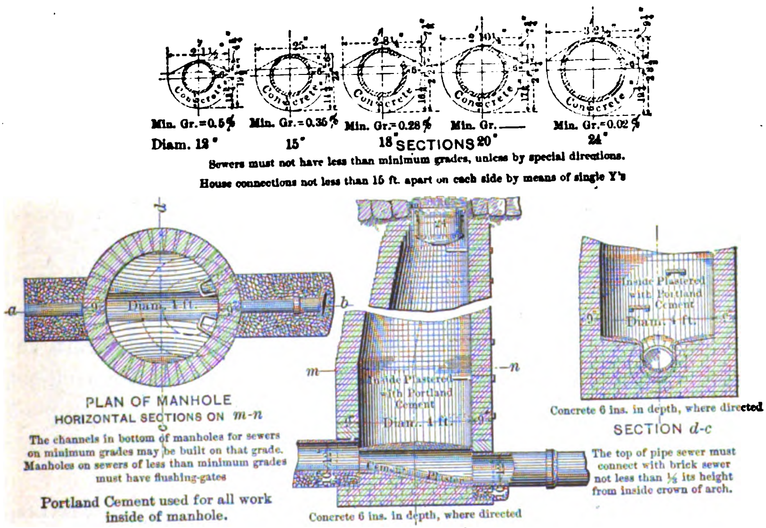
Total payment to contractor: $22 828.49.



GENERAL DETAILS FOR EGG-SHAPED SEWERS.

SCALE

FIG. 10.



GENERAL DETAILS FOR PIPE SEWERS.

SCALE

FIG. 11.

TABLE No. 7.—Prices Fixed in All Sewer Contracts.

Additional rubble masonry in place, per cubic yard, including all appurtenances	$4.50
Additional brick masonry, in place, per cubic yard	9.50
Additional manhole with cover, without bucket	25.00
Additional wellhole, per vertical foot	5.00
Each inlet with cover or grating, as follows:	
No. 1	96.00
No. 2	87.00
No. 3	81.00
No. 4	45.00
No. 5	40.00
Additional terra-cotta pipe not in concrete, per lineal foot	1.10
" " " " " " "	1.35
" " " " " " "	0.50

TABLE No. 8.—Summary of Sewer Work.

System.	Contract.	Shafts.	Total. length in feet.	Length in tunnel, in feet.	Total cost.
Twenty-fourth St. and Pennsylvania Ave....	No. 1	9	1 988.03	1 001.03	$92 586.42
Twenty-fourth St. and Pennsylvania Ave....	No. 2	16	3 470.66	2 878.02	126 496.15
Callowhill Street........	No. 1	8	3 094.61	1 751.87	80 342.75
Callowhill Street........	No. 2	12	2 953.20	2 953.20	69 662.71
Callowhill Street........	No. 3	9	2 467.97	2 060.97	35 976.81
Twelfth Street........	....	....	1 398.00	....	22 823.49
		54	15 372.47	10 645.09	$427 888.33

General Sewer Notes.

Except on Twelfth St., Buttonwood St. and Callowhill St. west of Twenty-second St., no provision was made in the sewers for house connections, on account of the depth of the main sewer beneath the street. Where it was necessary to provide for such connections, auxiliary pipe sewers were constructed at a higher elevation, and these emptied into the deep sewers through wellholes at suitable intervals.

In many cases, where high-level sewers existed already, it was posible to maintain them with their house connections intact by building

PLATE XXI.
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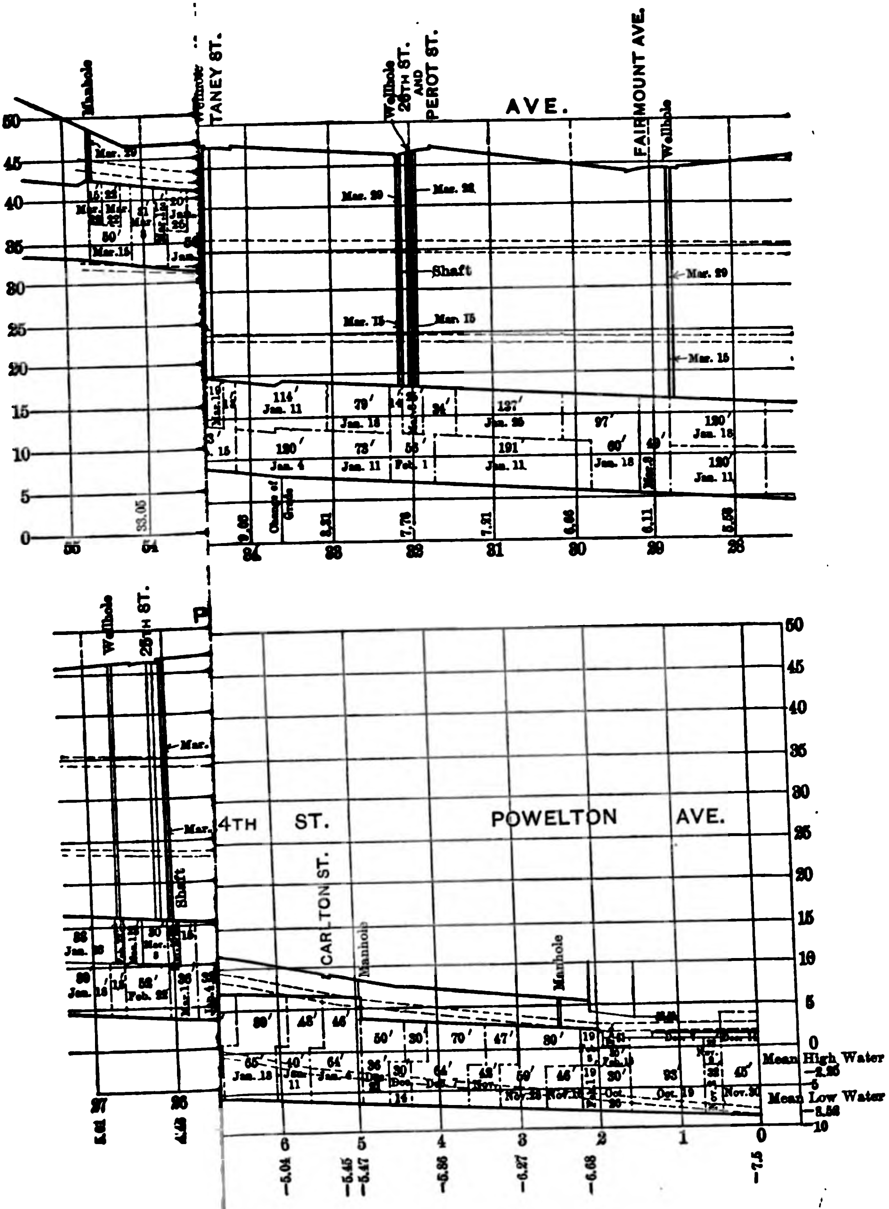


EN.

the new sewer in tunnel and not disturbing the surface of the street. Where it was necessary to give both old and new sewers the same alignment, side shafts were sunk, or the old sewer was carried across the shaft in a flume during construction. Wherever practicable, the shafts were sunk where manholes were required, thus accomplishing a double purpose.

The details of manholes, wellholes, etc., are shown in Figs. 8 to 11, which explain themselves sufficiently.

Plate XX shows the weekly progress made in excavation in tunnel and open cut on the Twenty-fourth St. and Pennsylvania Ave. System, and Plate XXI shows the weekly progress made in laying the brick work on the same system. As this work was pushed rapidly, the data may prove of interest. Work was always carried on in the tunnel excavation practically at night. Brickwork was only laid by day.

Engineering.

Under the ordinance authorizing this work, the Director of the Department of Public Works, who has supervision of the Engineering Bureaus of the City, is authorized and directed to appoint such assistant engineers, draughtsmen and inspectors as may be necessary for the efficient execution of the works.

The work was placed in charge of George S. Webster, M. Am. Soc. C. E., Chief Engineer, Bureau of Surveys, which Bureau has jurisdiction over such work, under the Department of Public Works. Mr. George E. Datesman is the Principal Assistant Engineer of this Bureau.

In June, 1894, Samuel Tobias Wagner, M. Am. Soc. C. E., was appointed First Assistant Engineer in charge of the work, and Mr. Charles H. Swan, Chief Draughtsman. Mr. Swan had for some time previously been engaged upon the preparation of the plans. In August, 1894, Mr. Richard L. D. Ashbridge, was appointed Second Assistant Engineer in charge of the work of construction of the sewers. Mr. Charles H. Ott, with a corps of assistants, was delegated to assist in giving lines and grades.

Owing to the difficulties of giving the lines and grades on this work, and the rapidity with which it was carried on at a large number of points at the same time, and on account of the uniformly good

results obtained, the greatest credit is due to the engineers in the field.

Under the ordinance, the Director of the Department of Public Works was directed to confer during the progress of the work with an engineer to be appointed by the Philadelphia and Reading Railroad Company, in regard to carrying out the specifications and securing the proper performance of the contracts. On March 30th, 1894, the late John A. Wilson, M. Am. Soc. C. E., was appointed Consulting Engineer to represent the Railroad Company. After Mr. Wilson's death, which occurred on January 19th, 1896, Joseph M. Wilson, M. Am. Soc. C. E., was appointed Consulting Engineer. Mr. John A. Wilson was closely identified with the preparation of the studies and the details of the work, with special reference to the railroad interests.

Vol. XXVI. FEBRUARY, 1900. No. 2.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

THE ALBANY WATER FILTRATION PLANT.

Discussion.*

By Messrs. George I. Bailey, W. B. Fuller, P. A. Maignen, George Hill, A. M. Miller, Rudolph Hering, William P. Mason, C. E. Fowler, George W. Fuller, George C. Whipple and George W. Rafter.

George I. Bailey, M. Am. Soc. C. E.—Two filters were put in service July 27th. Four more were started July 28th. All of these ran until August 9th, and three of them continued until August 12th. They were started with the hope of continuing, but the conditions were unfavorable in that the water was pumped direct to the filters, as the sedimentation basin was not ready for use; the court between the filters, in which scraped sand was to be deposited, washed and stored, was neither leveled nor paved; and the water in the river was roiled and disturbed by the contractors' operations, and particularly with the wash-water from the sand being prepared for the remaining two filters. The run of the filters was, therefore, stopped and not again commenced until September 5th, since which time their operation has been continuous. Mr. Bailey.

*This discussion (of the paper by Allen Hazen, Assoc. M. Am. Soc. C. E., printed in the *Proceedings* for November, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to March 30th, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Bailey.

Cost of Operation.

The work was organized as follows:

Filter operation:	10 laborers,	at	$1.50	per day.
	1 foreman	"	2.75	"
	1 watchman	"	1.50	"
	1 chemist	"	1 000.00	per year.
Pumping Station:	3 engineers	"	75.00	per month.
	3 firemen	"	60.00	"

The working day is eight hours for laborers, engineers and firemen, and over-time is paid for at the rates named. Occasionally, extra help has been hired, and paid for at these rates.

The gross cost of operation, including payroll, tools which are still in use, repairs, supplies of all kinds, wash-water, etc., etc., for the period from September 5th to December 25th, inclusive, 118 days, was $6 164.94. In this time 1 470 000 000 galls. were filtered, making an average of $4.19 per million gallons delivered from the filters.

The master mechanic of the works gives the following statement from his records, as the daily cost at the pumping station:

3 engineers.......................at	$2.48	$7.44
3 firemen........................"	1.98	5.94
3 tons coal ...."	2.72	8.16
1 laborer.........................."	1.50	1.50
9 galls. engine oil ...."	0.09	0.81
2 galls. cylinder oil ...."	0.11	0.22
5 galls. kerosene oil ...."	0.10	0.50
5 lbs. waste ...."	0.07	0.35
Steam packing, sheet rubber, soap, soda, mops, cloths, etc.		6.58
Total............................		$31.50

This makes the average cost of pumping $2.52 per million gallons received from the filters, and leaves $1.67 as the cost of operating the filters, including laboratory work. The cost of scraping, wheeling out, washing and replacing sand for the actual number of hours, and exclusive of superintendence, laboratory work, lost time, tools, etc., is $1.19 per million gallons treated.

Mr. Bailey.

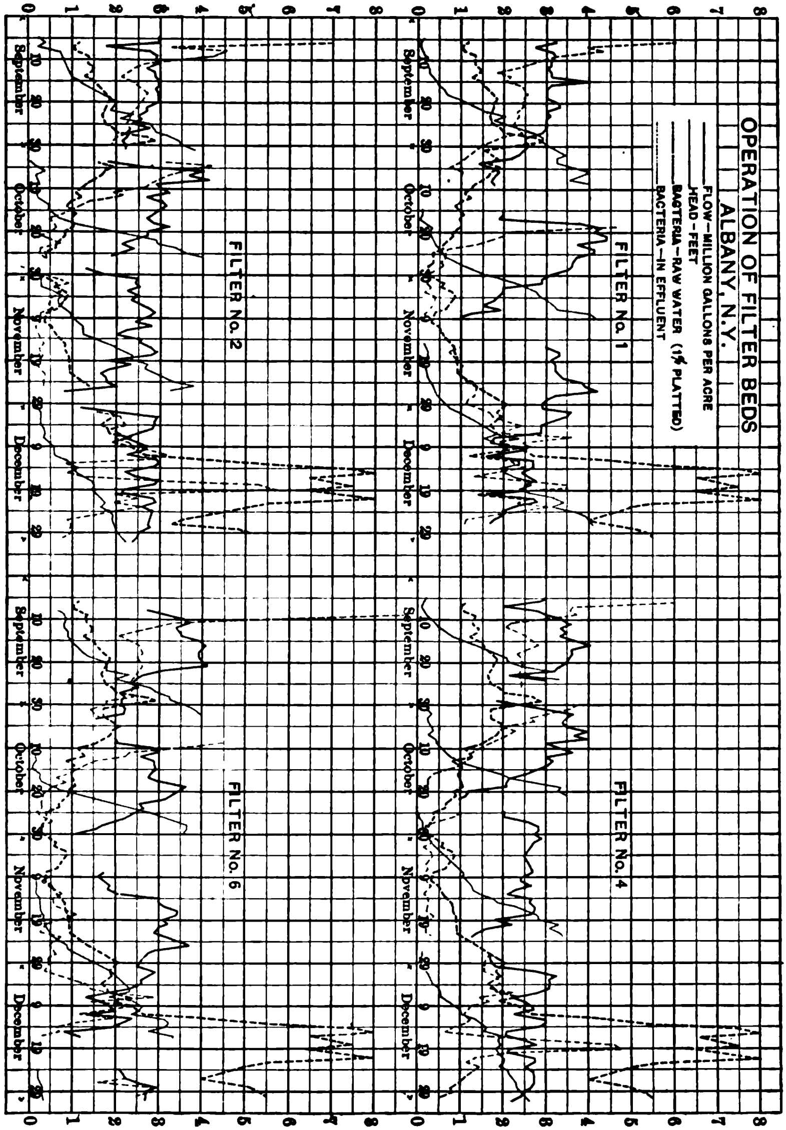


FIG. 16.

Mr. Bailey. TABLE No. 4.—GALLONS FILTERED AND HOURS EMPLOYED IN SCRAPING FILTERS AND WHEELING OUT SCRAPED SAND.

Filter No.	SERVICE.	Gallons filtered. (5% to be added for error.)	SCRAPING.		WHEELING OUT.	
	Hours		Hours.	Square yards.	Hours.	Barrows.
1............	266.45	22 409 000	43	Each scraping covers an area of 3 402 sq. yds.	86	983
1............	791.55	59 164 000	50		80	813
1............	559.25	41 457 000	45		81	789
1............	1 015.50	72 685 000	42		88	982
2............	291.00	26 581 000	46		115	1 027
2............	638.00	49 571 000	50		86	962
2............	534.00	44 822 000	47		79	771
2............	685.00	45 520 000	41		88	886
3............	342.75	32 377 000	57		91	1 026
3............	639.00	49 296 000	49		75	875
3............	579.20	42 296 000	44		82	726
3............	877.00	63 406 000	46		92	1 067
4............	355.10	32 357 000	52		124	1 119
4............	470.20	44 290 000	56		91	942
4............	548.00	45 757 000	49		81	763
4............	718.50	50 717 000	46		81	878
5............	664.60	53 359 000	51		78	820
5............	556.70	40 428 000	27		86	874
5............	840.00	60 459 000	43		83	802
6............	312.70	28 826 000	53		109	1 007
6............	618.70	55 263 000	51		78	836
6............	547.00	41 133 000	44		76	808
6............	982.00	65 457 000	44		86	920
7............	631.00	38 506 000	42		69	835
8............	361.20	33 006 000	56		105	1 105
8............	513.00	40 497 000	53		76	987
8............	591.00	50 529 000	50		79	805
8............	807.20	56 150 000	40		78	819
28............	16 685.60	1 286 268 000	1 317	95 256	2 420	25 179

The filters have been cleaned 26 times in all, up to December 25th, or a little more than three times each. The total amount of sand treated, as measured when replaced, was 850 cu. yds. From the books of the foreman, the following records are taken:

Scraping.—88 452 sq. yds. = 18.3 acres; time, 1 227 hours = 67 hours per acre.

Wheeling Out Scraped Sand.—23 180 barrows, 2 235 hours = 27.3 barrows per cubic yard = 0.38 cu. yd. per hour. The average length of wheel, going and coming, was 600 lin. ft. = 1.18 miles per man per hour.

Washing.—18 262 barrows, 2 068 hours, 21.5 barrows per cubic yard = 0.41 cu. yd. per hour.

From experiments made by John H. Gregory, Jun. Am. Soc. C. E., who was Resident Engineer for Mr. Hazen during the completion of the work, the speaker is informed that the volume of water for washing the sand varied from 12 to 14 times the volume of sand washed. In the cost of operation the volume has been estimated at 15, at a cost of $0.04 per thousand gallons.

Refilling.—18 550 barrows, 1 630 hours, 21.8 barrows per cubic yard = 0.52 cu. yd. per hour. This work was chiefly done by extra labor.

The average depth of scraping was about ⅜ in., computed from the total quantity of sand replaced and the area scraped. Mr. Bailey.

During the periods covered by these scrapings, the filters yielded 1 212 000 000 galls., an average of 66 600 000 galls. per acre between scrapings. This includes the first run of the filters, when the unnaturally turbid water already mentioned was pumped directly on the beds.

Bacteria.

Table No. 5 shows the weekly averages of the bacterial removal:

TABLE No. 5.

Week ending	Bacteria Per Cubic Centimeter.		Percentage of removal.
	Unfiltered.	Filtered.	
September 9	11 545	608	94.8
16	14 068	306	97.8
23	17 480	273	98.5
30	22 600	259	98.8
October 7	18 766	250	98.7
14	11 783	178	98.5
21	9 988	85	99.1
28	4 783	84	98.4
November 4	6 091	56	99.1
11	5 141	46	99.1
18	7 950	69	99.1
25	11 090	79	99.3
December 2	19 240	109	99.4
9	20 016	198	99.1
16	57 700	142	99.7
23	66 000	327	99.5
30	48 940	215	99.6

Table No. 6 shows in detail the result obtained from each filter each day. It will be noted that the percentage of removal is high, and that the bacterial count in the filtered water is low.

Color.

The average color of the Hudson River water corresponds to 0.50 to 0.60 on the platinum scale, and about 40% of this color is removed from the water by the filters.

Turbidity.

In periods of freshet the water is very turbid. The highest turbidity reached since the operation of the filter was in December, when the raw water showed 0.60. The effluent then contained 0.008. Generally the raw water runs about 0.035, all of which is removed. The platinum-wire standard is used.

Mr. Bailey. TABLE No. 6.—ALBANY FILTER PLANT.—BACTERIAL REMOVAL BY FILTERS.

Day of month.	Raw Water	1		2		3		4		5		6		7		8		Pure Water.	
	Bacteria.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.	Bacteria.	Per cent. removed.
	September.																		
1	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
2	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
3	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
4	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
5	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	800	....
6	11 500	600	94.8	700	93.9	500	95.7	600	94.8	3 700	67.8	....	....	....	....	....	....	800	93.0
7	10 200	392	96.2	331	96.8	397	96.3	361	96.5	1 800	82.4	....	....	....	....	420	95.9	508	95.0
8	10 860	432	96.1	457	95.8	363	96.7	355	96.7	913	91.6	1 100	89.9	....	....	437	96.0	571	94.7
9	13 700	337	96.4	431	96.8	381	97.2	372	97.3	731	94.7	873	93.7	....	....	541	96.8	567	95.9
10	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
11	13 200	256	98.1	265	97.9	353	97.3	220	98.3	433	96.7	336	97.5	....	....	194	98.5	420	96.8
12	14 700	231	98.4	248	98.3	320	97.8	207	98.6	440	97.0	289	98.0	....	....	214	98.5	294	96.9
13	14 300	182	98.7	....	....	321	97.8	228	98.4	402	97.2	243	98.3	....	....	220	98.5	284	98.0
14	12 400	218	98.2	211	98.3	....	....	256	98.0	550	95.6	205	98.4	....	....	197	98.4	257	97.9
15	14 700	206	98.6	237	98.4	....	....	273	98.1	362	97.5	228	98.4	....	....	200	98.6	286	98.1
16	15 200	242	98.4	247	98.4	....	....	286	98.1	375	97.5	251	98.3	....	....	216	98.6	298	98.2
17	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
18	17 100	256	98.5	276	97.4	....	....	232	98.6	360	97.9	261	98.5	....	....	231	98.7	289	98.3
19	18 900	247	98.7	263	98.6	....	....	241	98.7	345	98.2	250	98.7	....	....	221	98.8	273	98.6
20	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
21	17 600	243	98.6	251	98.6	....	....	249	98.6	336	98.1	261	97.5	....	....	232	98.7	265	98.5
22	17 100	231	98.6	243	98.6	625	96.3	241	98.6	312	98.2	254	98.6	....	....	245	98.6	276	98.4
23	16 700	236	98.6	232	98.6	351	97.9	237	98.6	289	98.2	246	98.5	....	....	231	98.6	261	98.4
24	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
25	17 400	221	98.7	224	98.7	316	98.2	246	98.6	261	98.5	231	98.7	....	....	219	98.7	250	98.6
26	19 300	231	98.8	237	98.8	320	98.3	240	98.8	275	98.6	239	98.8	....	....	244	98.7	267	98.6
27	20 200	241	98.8	229	98.9	276	98.6	....	....	261	98.7	233	98.8	....	....	237	98.8	257	98.7
28	27 100	197	99.3	205	99.2	267	99.0	....	....	247	99.1	241	99.1	....	....	....	....	261	99.0
29	29 300	172	99.4	191	99.3	240	99.2	....	....	215	99.3	176	99.4	....	....	....	....	244	99.2
30	22 300	258	98.8	189	99.2	236	98.4	375	98.3	237	98.9	179	99.2	....	....	700	96.9	278	98.8
	October.																		
1	17 000	134	99.3	....	....	215	98.7	360	97.9	155	99.1	137	99.2	....	....	390	97.7	267	98.4
2	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
3	17 200	109	99.4	....	....	194	98.9	265	98.5	....	....	162	99.1	....	....	420	97.6	230	98.7
4	19 400	98	99.5	313	98.4	180	99.1	278	98.6	....	....	....	....	....	....	390	98.0	250	98.7
5	21 000	71	99.7	421	98.0	142	99.3	191	99.1	....	....	....	....	....	....	340	98.4	240	98.9
6	19 200	105	99.5	360	98.2	150	99.2	230	98.8	640	96.7	....	....	....	....	280	98.6	264	98.7
7	18 800	102	99.5	325	98.3	140	99.3	219	98.8	560	97.0	....	....	....	....	260	98.6	250	98.7
8	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
9	15 100	....	....	275	98.2	110	99.3	170	98.9	260	98.3	450	97.7	....	....	220	98.5	239	98.4
10	11 300	....	....	265	97.7	120	98.9	143	98.7	240	97.9	340	97.0	....	....	156	98.6	233	97.9
11	12 100	....	....	197	98.4	90	99.3	125	99.0	206	98.3	231	98.1	....	....	144	98.8	170	98.6
12	13 200	....	....	210	98.4	83	99.4	130	99.0	212	98.4	220	98.3	....	....	170	98.7	178	98.7
13	8 200	....	....	140	98.4	60	99.3	119	98.7	165	98.0	155	98.2	....	....	110	98.8	135	98.5
14	10 800	....	....	115	98.9	39	99.6	106	99.0	135	98.8	130	98.8	....	....	87	99.2	115	98.9
15	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
16	9 800	....	....	72	99.3	36	99.6	39	99.6	86	99.1	91	99.1	....	....	34	99.7	79	99.2
17	10 200	....	....	45	99.6	50	99.5	27	99.7	71	99.3	60	99.4	....	....	35	99.7	50	99.5
18	10 400	....	....	53	99.5	72	99.3	26	99.8	32	99.7	86	99.2	....	....	25	99.8	46	99.6
19	11 000	460	95.8	61	99.7	....	....	24	99.8	74	99.3	54	99.5	....	....	43	99.6	160	98.5
20	8 800	198	97.8	50	99.4	....	....	27	99.7	121	98.6	37	99.6	....	....	41	99.5	76	99.1
21	9 400	200	97.8	24	99.8	....	....	70	99.3	32	99.7	30	99.7	....	....	33	99.7	101	98.9
22	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
23	6 900	135	98.1	66	99.0	141	98.0	35	99.5	35	99.5	27	99.6	361	94.8	15	99.8	125	98.2
24	7 000	70	99.0	24	99.7	125	98.2	....	....	36	99.5	30	99.6	471	93.3	26	99.6	120	98.3
25	3 900	66	98.3	22	99.6	70	98.2	105	97.3	36	99.1	16	99.6	250	93.6	....	....	70	98.2
26	4 200	69	98.4	33	99.2	63	98.5	87	97.9	30	99.3	26	99.4	125	97.0	....	....	75	98.2
27	3 500	59	98.3	....	....	64	98.2	36	99.0	24	99.3	54	98.5	100	97.2	64	98.2	58	98.4
28	2 900	62	97.9	....	....	44	98.5	20	99.3	14	99.5	36	98.8	165	94.3	58	98.0	57	98.0
29	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
30	2 500	27	98.9	31	98.8	18	99.3	14	99.4	....	....	17	99.3	150	94.0	16	99.4	37	98.5
31	4 500	24	99.5	68	98.5	26	99.4	34	99.3	....	....	....	....	165	96.3	34	99.3	68	98.5

TABLE No. 6—(*Continued*).

Mr. Bailey.

Day of month.	Bacteria.	1 Bacteria.	1 Per cent. removed.	2 Bacteria.	2 Per cent. removed.	3 Bacteria.	3 Per cent. removed.	4 Bacteria.	4 Per cent. removed.	5 Bacteria.	5 Per cent. removed.	6 Bacteria.	6 Per cent. removed.	7 Bacteria.	7 Per cent. removed.	8 Bacteria.	8 Per cent. removed.	Pure Water. Bacteria.	Pure Water. Per cent. removed.
	November.																		
1	6 350	23	99.6	96	98.5	24	99.6	32	99.5	....	....	....	....	175	97.1	22	99.6	53	99.2
2	5 700	31	99.4	53	99.1	23	99.6	14	99.8	93	98.4	....	....	150	97.4	25	99.6	51	99.1
3	8 500	15	99.8	72	99.2	29	99.7	31	99.6	140	98.4	....	....	L	....	27	99.7	57	99.3
4	9 000	48	99.5	54	99.4	22	99.8	38	99.6	120	98.7	....	....	172	98.1	97	98.9	69	99.2
5	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
6	7 750	34	99.6	43	99.5	19	99.8	22	99.7	61	99.2	....	....	95	98.8	18	99.8	59	99.2
7	6 900	25	99.6	57	99.2	26	99.6	27	99.6	128	98.2	....	....	115	98.3	17	99.8	52	99.3
8	4 000	16	99.6	26	99.4	20	99.5	L	....	90	97.8	....	....	76	98.1	18	99.6	48	98.8
9	3 800	19	99.5	25	99.4	28	99.3	11	99.7	40	99.0	50	98.7	88	97.7	15	99.6	40	99.0
10	3 400	....	....	26	99.2	19	99.5	22	99.4	36	98.9	28	98.2	120	96.5	17	99.5	35	99.0
11	5 000	....	....	16	99.7	14	99.7	33	99.3	62	98.8	35	99.3	104	97.9	15	99.7	44	99.1
12	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
13	6 200	....	....	20	99.7	12	99.8	17	99.7	41	99.3	76	98.8	65	98.9	27	99.6	45	99.3
14	7 350	....	....	26	99.7	17	99.8	25	99.7	38	99.5	57	99.2	89	98.8	35	99.5	57	99.2
15	8 200	....	....	33	99.6	29	99.7	28	99.7	45	99.5	55	99.3	96	98.8	37	99.6	65	99.2
16	8 200	98	98.8	25	99.7	....	....	23	99.7	31	99.6	63	99.2	119	98.1	27	99.7	87	98.6
17	8 500	75	99.1	27	99.7	....	....	23	99.7	40	99.5	85	99.0	92	98.9	32	99.6	75	99.1
18	9 250	85	99.1	31	99.7	120	98.7	41	99.6	36	99.6	112	98.8	....	....	23	99.8	83	99.2
19	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
20	9 200	107	98.8	15	99.8	135	98.5	21	99.8	28	99.7	40	99.6	....	....	18	99.8	87	99.1
21	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
22	9 500	95	99.0	35	99.6	120	98.7	23	99.8	19	99.8	24	99.8	....	....	17	99.8	85	99.1
23	10 750	115	98.9	36	99.7	105	99.0	40	99.6	28	99.7	73	99.3	....	....	28	99.7	89	99.2
24	12 000	130	98.9	32	99.7	122	99.0	....	....	30	99.8	57	99.5	....	....	32	99.7	101	99.2
25	14 000	140	99.0	32	99.7	150	98.9	....	....	55	99.6	48	99.6	....	....	29	99.8	112	99.2
26	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
27	17 300	135	99.2	....	....	124	99.2	175	99.0	33	99.8	64	99.7	....	....	57	99.8	115	99.3
28	19 400	118	99.4	....	....	108	99.4	160	99.2	35	99.8	44	99.8	....	....	27	99.9	97	99.5
29	21 000	115	99.4	....	....	100	99.5	....	....	25	99.9	50	99.8	....	....	39	99.8	99	99.5
30	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
	December.																		
1	18 600	165	99.1	153	99.2	115	99.4	159	99.2	20	99.9	33	99.8	....	....	....	....	107	99.4
2	19 900	179	99.1	210	99.1	185	99.1	187	99.1	56	99.7	64	99.7	....	....	....	....	130	99.3
3	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
4	16 800	245	98.6	223	98.7	256	98.5	195	98.8	110	99.3	136	99.2	....	....	153	99.1	185	98.9
5	18 100	215	98.8	187	98.9	235	98.7	173	99.0	105	99.4	145	99.2	....	....	225	98.8	195	98.2
6	18 500	190	99.0	210	98.9	225	98.8	230	98.8	....	....	185	99.0	....	....	245	98.6	200	98.9
7	23 900	360	98.5	197	99.2	225	99.1	144	99.4	....	....	290	98.8	....	....	265	98.9	219	99.1
8	19 800	245	98.7	128	99.3	212	99.9	172	99.1	....	....	145	99.2	....	....	360	98.2	182	99.1
9	23 500	270	99.2	140	99.4	276	98.8	128	99.5	....	....	246	99.0	....	....	340	98.6	210	99.1
10	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
11	27 000	120	99.6	320	98.8	115	99.6	110	99.6	233	99.1	116	99.6	....	....	310	98.8	152	99.5
12	44 000	213	99.5	285	99.4	117	99.7	128	99.7	249	99.4	198	99.6	....	....	276	99.4	192	99.6
13	55 000	225	99.6	88	99.8	169	99.7	116	99.8	255	99.6	110	99.8	....	....	256	99.5	163	99.7
14	75 000	249	99.7	112	99.9	64	99.9	85	99.9	216	99.7	62	99.9	....	....	211	99.7	96	99.0
15	80 200	185	99.8	87	99.9	182	99.8	68	99.9	308	99.6	72	99.9	....	....	198	99.8	115	99.9
16	65 000	130	99.8	136	99.8	210	99.7	165	99.7	378	99.4	30	99.9	....	....	147	99.8	131	99.8
17	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
18	75 000	350	99.5	525	99.3	450	99.4	460	99.4	575	99.2	....	....	Lo	st	560	99.2	435	99.4
19	64 000	350	99.5	570	99.1	490	99.2	475	99.3	520	99.2	....	....	315	99.5	470	99.3	450	99.3
20	71 000	137	99.8	195	99.7	243	99.7	210	99.7	320	99.5	....	....	312	99.6	350	99.5	275	99.6
21	80 000	268	99.7	250	99.7	325	99.6	135	99.8	360	99.5	....	....	450	99.4	330	99.6	310	99.6
22	56 000	233	99.6	183	99.7	260	99.5	114	99.8	310	99.4	....	....	475	99.2	305	99.5	291	99.5
23	50 000	310	99.4	255	99.5	257	99.5	146	99.7	175	99.6	....	....	Lo	st	150	99.7	205	99.6
24	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
25	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....
26	40 000	115	99.7	84	99.8	....	....	103	99.7	117	99.7	225	99.4	315	99.2	140	99.7	162	99.6
27	45 000	110	99.7	98	99.8	....	....	95	99.8	100	99.8	160	99.6	340	99.2	90	99.8	180	99.6
28	51 000	....	....	95	99.8	490	99.0	78	99.8	150	99.7	280	99.5	370	99.3	119	99.8	250	99.5
29	58 700	....	....	100	99.8	370	99.4	85	99.8	141	99.7	290	99.5	325	99.4	115	99.8	255	99.5
30	55 000	....	....	80	99.9	380	99.3	55	99.9	112	99.8	250	99.5	280	99.5	125	99.8	230	99.5
31	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....	....

Typhoid Fever.

Mr. Bailey. The reason for building the filters was the sewage pollution of the Hudson River and the large death rate from typhoid fever. The average number of deaths from this cause for the nine years ending with 1898 was 85 per annum. During the four months in which the filters have been in operation seven deaths from this cause have been reported. For the corresponding months of the nine years ending with 1898 the average number has been 24. The deaths from this cause have thus been reduced in the ratio of 24 to 7, and one of these seven was in a family which did not use city water.

Mr. W. B. Fuller. William B. Fuller, M. Am. Soc. C. E.—It has occurred to the speaker that it would interest the members if a few words were added concerning the details of construction, of the Albany Filter Plant, which present special features.

The sedimentation basin is situated close to the river bank and lies almost wholly in embankment above the natural surface of the ground. The embankments were constructed of the best available materials, and with great care, so as to minimize any effects of after-settling; but, for tightness, reliance was placed on a 16-in. layer of puddle, which covered the entire bottom of the basin and extended up the sides to 1 ft. above high-water level. For a distance of 5 ft. below high-water level, or to below the lowest level to which the basin is likely to be drawn in winter, the puddle is set back 2 ft. 10 ins. from the face so as to be beyond injury from frost, and is covered with a 2-ft. layer of gravel and 10 ins. of blue limestone paving. In all other places the puddle is faced with 6 ins. of concrete, to prevent mechanical injury to its surface and to present an easily cleansed and smooth surface for the interior of the basin.

The materials of the puddle were equal parts of clay and a sandy gravel containing about 40% sand and 60% gravel of all sizes to about 1 in. in diameter. Clean gravel of a size from ¼ to 1 in. was tried, but was not a success. These materials were mixed in a screw-puddle, continuous mixer, such as is often used for mixing concrete, water being added and the mixing continued until all the clay lumps were softened and the clay had penetrated thoroughly all the interstices of the gravel.

In this plastic condition the puddle was placed in position in 6-in. layers over a larger surface and left to dry out. In the process of drying, a large number of shrinkage cracks appeared throughout the mass, but by thorough ramming these cracks were closed and the entire mass consolidated. This process was continued for each of the three layers. The idea was, that with the shrinkage cracks closed and the excess water removed, there was no further tendency to shrinkage, and any water then entering the puddle would expand it and close any remaining cracks. Some of this puddle stood uncovered on 1 on 1½

slopes for over a year, exposed to rain and frost, but no cracking or deterioration of any kind was noticed, and it remained as hard as hard-pan earth. Mr. W. B. Fuller.

The paving of the upper sides of the sedimentation basin is of blue limestone blocks, rather larger than the usual size, being about 10 to 15 ins. deep, 15 to 36 ins. long and 8 to 20 ins. wide. Two masons and one helper together would lay about 16 sq. yds. per day, and the labor cost of laying the stone and gravel, including the teaming of the materials about 800 lin. ft., was $0.72 per square yard.

The gravel used in the joints and under the paving was the waste from the filter-sand screen. It was perfectly clean, of sizes from ¼ to to 1 in., the largest amount being about ⅝ in., and made an ideal material for the purpose.

The piping about the filters and sedimentation basin was all of light-weight cast-iron pipe, with hub and spigot, lead-caulked joints. The entire system was laid at the same time, the trenches and all bell holes being left open, and the joints made water-tight under a hydrostatic pressure of 50 lbs. per square inch, before making any refill.

All the concrete used in the floors, walls and vaulting, amounting to 22 100 cu. yds., was machine-mixed, especial care being taken with the mixing and placing. A mixed sand and gravel was obtained from the river by dredging, and was brought to and deposited near the mixers, and then washed and screened into three sizes—sand, gravel and tailings. The sand was of very good quality, sharp and clean. The gravel was of irregular, smooth-edged stone from ¼ to 1½ ins. in diameter, but varying greatly as to average size from day to day, sometimes fine and sometimes coarse. The tailings were passed through a stone crusher and broken to sizes ranging from ¾ to 2½ ins. in diameter.

Mechanical analyses of the sizes of these three materials were made daily or oftener, and from these analyses the proper proportions of a mixture of the three to give a minimum number of voids was deduced. The total quantity of the three materials used with a unit quantity of cement was always constant, but by thus varying the proportions of the ingredients themselves, a very uniform concrete product was obtained, independent of the variation in the average size of any particular ingredient. The proportions ordinarily followed were 1 part of cement, 3 parts of sand, 4 parts of gravel, and 1 part of crushed stone.

The mixing was done in a cubical mixer, to which a measured quantity of water could be introduced after the materials had been thoroughly mixed dry. With the apparatus as used at Albany, the concrete was always mixed properly and of the right consistency; half an hour's attention to it, when starting, insuring uniformity for all day unless there were great weather changes, in which case the quantity of water had to be changed more frequently.

Mr. W. B. Fuller.

The transportation of the concrete from the mixer and its deposition in place in the work, which had been described by the author, was ideal, from an engineering point of view. From the time of taking the concrete from the mixer until it was put in place 800 ft. distant the interval of 5 minutes was not uncommon, and the average time did not exceed 15 minutes.

The cost of labor and coal for measuring quantities, and mixing, loading, transporting and tamping concrete, during an average of about three months of the best organization, was about as follows:

Measuring, mixing and loading....	$0.20	per cubic yard.
Transporting by rail and cables....	0.12	" "
Laying and tamping on vaulting...	0.14	" "
Laying and tamping floors and walls, including setting forms...	0.22	" "

These prices do no not include general superintendence, profit or cost of machinery.

The design of the vaulting, as has been stated by the author, is in the form of an elliptical groin with dimensions as follows: span, 12 ft.; rise, 2 ft. 6 ins.; thickness at crown, 6 ins.; thickness over center of pier, 2 ft. 6 ins. This work was figured for strength according to the theory of the arch, assuming that brick masonry and concrete spandrel filling were to be used. The material actually used was Portland cement concrete laid as a monolith with the pier in the center, and, as thus constructed, it is exceedingly doubtful if there is any arch action whatever in the structure. From recent observations and from some tests made on small models the speaker believes that such a groined arch acts in tension as an inverted pyramidical dome. If this is the case, the depression over the piers could be increased materially and the cost of the vaulting reduced. Even as constructed, the adoption of the 6-in. depression over each pier saved 1 053 cu. yds. of concrete, which would have cost $6 560 at contract prices.

The speaker wishes to controvert an impression, which seems to be prevalent, that permanent masonry vaulting is very expensive, several recent estimates placing the cost of covered filters at from 50% to 100% in excess of the cost of open filters. The total extra cost of the vaulting at Albany, including extra thickness of floors, piers, roof drains, manholes, sand-run entrances, earth covering, etc., at the contract prices, was approximately $0.315 per square foot of area inside the walls, or $13 700 per acre, while the total cost of the filters was about $45 600 per acre; that is, the vaulting represents only about 30% of the cost of the filters.

The lumber of the centering for the vaulting was of spruce for the ribs and posts, and of hemlock for the lagging, which was 1½ and 3 ins. wide and 1 in. thick. The entire centering for two filters was formed by machinery, the ribs put together to a template, and the lagging all

sawed to proper lengths and bevels. For the first two filters the centers were put together in place, and were so constructed that when struck they would come down in sections and could be moved forward and used in the corresponding bay of the next filter. Mr. W. B. Fuller.

The total cost of the centering was approximately as follows:

Building centers covering 62 560 sq. ft.					
Labor—					
Foreman,	435 hours,	at $0.35		$ 152.25	
Carpenters,	4 873 "	0.225		1 096.42	
Laborers,	3 447 "	0.15		517.05	
Painters,	577 "	0.15		86.55	
Teaming,	324 "	0.40		121.60	
				$1 973.87	
Materials—					
Lumber,	313 000 ft., B. M.		$5 700.00		
Nails,	3 700 lbs.		111.00		
Tar,	8 bbls.		24.00		
				5 835.00	
					$7 808.87
Taking down, moving and putting up centers covering 196 660 sq. ft.					
Labor—					
Foreman,	2 359 hours,	at $0.35		$ 825.65	
Carpenters,	12 766 "	0.225		2 872.35	
Laborers,	24 062 "	0.15		3 609.30	
Team,	430 "	0.40		172.00	
				$7 479.30	
Materials—					
Lumber,	3 000 ft., B. M.		$60.00		
Nails,	3 000 lbs.		90 00		
				$150.00	
Total				$7 629.30	
					7 629.30
Total approximate cost					$15 438.17

The total area centered was 259 220 sq. ft., and the average cost per square foot was 6 cents, to which should be added general superintendence and a reasonable profit.

As bearing upon the need of a covering, for protection against frost, the following records for Albany are quoted. The temperature of the air varies from + 98° to — 18° Fahr. per annum, the average temperature for the year being + 45° Fahr. These figures are the result of 22 years' observation by the local weather bureau. The river is frozen over from December 16th to March 19th, an average length

Mr. W. B. Fuller. of 93 days, as shown by an average of the records of 75 years. The longest time in any one season in which the river has remained frozen over was 117 days.

There are other advantages, besides the prevention of the formation of ice, which enable the covered filter to be operated at a lower cost for maintenance than the open filter, a few of which are as follows:

By preventing the formation of algæ growths.

By maintaining the temperature of the water practically even, thus keeping the friction of the sand constant, as by maintaining uniform conditions, improvement of the filtrate results.

By preventing the action of the heat and the direct rays of the sun in summer from baking the surface during the operation of scraping, thereby occasioning the removal of a greater thickness of sand.

By keeping the surface from disturbance by wind and thus allowing more efficient sedimentation.

By preventing the fall of rain on the surface during cleaning, which causes compacting of the surface and necessitates re-raking.

By preventing the fall of snow on the surface during cleaning, which causes the removal of a greater thickness of sand.

By preventing the increase in cost of working sand containing leaves and algæ growths.

By admitting of uninterrupted scraping of the filter during all kinds of weather.

As the extra cost of vaulting is only about $13 700 per acre, which by added knowledge can possibly be reduced to $10 000 per acre in another plant, and which amount at 3% interest represents only $300 to $400 per year, it is seen that in many places other considerations besides that of protection from ice would lead to the use of covered filter beds.

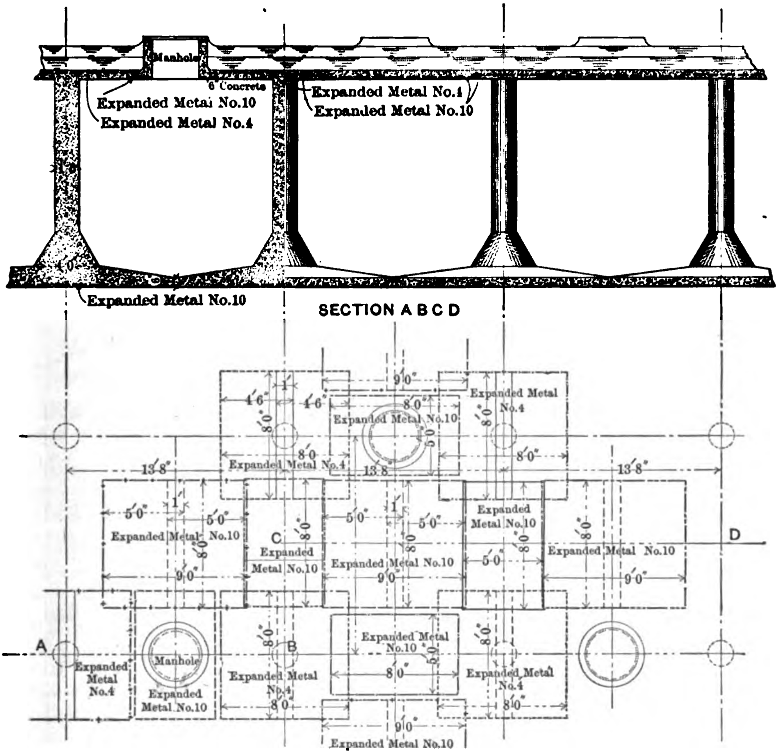
Mr. Maignen. P. A. Maignen, Assoc. Am. Soc. C. E.—The construction of the filters at Choisy-le-Roi, near Paris, is interesting. Some of the walls of the filters are very thin, being only 2.4 ins.; and their construction with what is termed in the United States "expanded metal" and concrete has been quite satisfactory. The company which operates these filters has built filter beds 120 ft. square, and others 60 ft. square. They found the smaller beds better for solidity and also because of the greater convenience in cleaning.

In the Albany plant the sand-washing apparatus is not covered. The speaker supposes that it is the intention to have it covered, in order that the washing of the sand may not be delayed by freezing weather.

The provision made for preventing the raw water, which may go down through or along the retaining walls, from mixing in an unpurified state with the filtered water, is interesting. It would seem better that such provision be made at the top of the sand bed instead of at the bottom.

George Hill, M. Am. Soc. C. E.—The paper is so completely what it should be, it describes so completely the way in which the work was done, its cost and its effect, that the speaker does not wish to be understood as offering any criticism in calling attention to a few details wherein the cost of the work might have been reduced, without in any way impairing its efficiency. Mr. Hill.

Referring to Fig. 7, it might be well to note that 8-in. beams are too shallow to be used with a projection of 5 ft., as they would be apt to



PLAN SHOWING DISTRIBUTION OF EXPANDED METAL

Fig. 17.

deflect so much as to crack the concrete, admitting moisture and therefore hastening decay. Fifteen-inch beams used in the grillage would have had five times the strength, with but two and one-half times the weight, would have had practically no deflection, a thicker section to resist rust, and, being spaced wider, would have given more room to pack the concrete between them.

The steel track used by the speaker, and described in a previous

Mr. Hill. paper,* would have been particularly applicable for the handling of dirty sand, as mentioned on page 768, and would have been found to be inexpensive. The speaker has used it for coal and ash tracks carrying loads of 500 lbs. very satisfactorily.

The principal feature wherein a saving could have been effected was in the floor and covering of the filtration basins. At the time when estimates were being made, one of the prospective bidders consulted with the speaker regarding the construction proposed and was advised to suggest the plan shown in Fig. 17.

It is probable that the lack of any published data regarding the action of steel and concrete in combination, and especially the lack of knowledge regarding the resistance of continuous slabs supported at a number of points, lead, in a measure, to the rejection of this advice, if it ever was presented by the bidder. Numerous experiments made by the speaker confirm him in the belief that the carrying capacity of a slab supported at a number of points and made continuous over them, is for a portion of the slab included between one set of supports four times as great as that of a slab of the same sectional area and dimensions supported at two opposite edges, or, for the sections proposed, 650 lbs. per square foot safe working load.

The relative costs for one bay are as follows:

As executed:

5.8 cu. yds. vaulting	$22.35
8.72 " flooring	20.15
1.24 " brick work	10.08
Total	$52.58

As proposed:

10 cu. yds. in roof slab, supporting column, floor and foundation	$23.10
Centering	7.47
Expanded metal	10.60
Total	$41.17

Saving 22 per cent.

It will be observed that the centering is far more simple, and the placing of the concrete less costly. The ramming of the concrete could be done with a roller instead of by hand, thus effecting a further saving. None of these points have been taken into account in the above comparison. The capacity of the filter could have been increased by raising the upper limit of the filling or the cost decreased by reducing the height of the pier and decreasing the depth of the excavation.

* *Transactions*, Am. Soc. C. E., Vol. xxxix, p. 620.

A. M. Miller, M. Am. Soc. C. E.—There appears to be some misapprehension as to the cost of concrete. The author states the price per cubic yard paid to the contractor, but the City of Albany furnished the cement. The cost mentioned by Mr. Hill, for the construction of the filters, is exclusive of the cost of cement. The author calculates that $1\frac{1}{4}$ bbls. of cement per cubic yard were required, and this, at $1.93\frac{1}{2}$ per barrel, would add \$2.42 to the cost per cubic yard. This must be borne in mind when examining the estimates. These questions, however, as to details of costs, strength of materials, etc., can be readily answered. Mr. Miller.

The speaker has heard that the Board of Health of New York City has stated that eventually the whole of the Croton water supply, some 300 000 000 galls. per day, must be filtered. The handling of such a large quantity will be a very serious question. The speaker is now considering the filtration of a supply of 60 000 000 galls. per day, and is glad to listen to those who have had some experience in these matters.

Information is needed as to methods of examining water, the quantity or thickness of sand required for filtering, the rate of filtration, etc., and Mr. Hazen's paper is very instructive in regard to these matters.

Mr. Bailey's statements in reference to the percentage of bacteria removed are interesting, though the speaker does not believe in expressing the results in that way. For instance, even though 95% of the bacteria may be removed by filtration, yet, possibly 700 bacteria come through, and thus the percentage method is misleading.

The cost of the filters per acre is an important item. From the statements of cost in the paper it is found that this was about \$45 600. In the recent report of the Board of Experts on the water supply of Philadelphia, the cost is stated as about \$37 000 per acre. Is the difference in cost due to the used of covered and open filters? If so, what is the estimated cost of the vaulting per acre?

Rudolph Hering, M. Am. Soc. C. E.—Albany has a raw water supply, as well as the filtered supply, and these two supplies are mixed and stored in an open reservoir. Mr. Bailey's statement in reference to the reduction of the typhoid fever death rate applies to the mixed waters. If this death rate should not be reduced as much as in other cities the filter should not be debited with the discrepancy, because raw water is added to the filtered water, and because both are stored in an open reservoir exposed to the air, dust, and other aerial contamination. Mr. Hering.

In Philadelphia it is proposed to cover all filters and reservoirs containing filtered water. The difference between the cost of the Albany filters and the estimated cost of the Philadelphia filters, is due to the difference in the prices of some of the materials, and some parts of the design. Otherwise there is no difference.

Mr. Hering. Mr. W. B. Fuller's description of the details of construction is interesting and valuable. He has pointed out very clearly the advantages of having covered, rather than open, filters, and the probable additional cost thereof. He has also pointed out the advantage of a batter on the sides to prevent the formation of seams which would allow raw water to pass through the filter more rapidly than permissible. An expedient which further helps to prevent an undue speed of the percolating water through the larger interstices necessarily formed between the sand and the smooth surface of the adjoining wall, is the transformation of this smooth surface into a sanded surface. This is done simply by first painting it with cement and then throwing sand against it. The sand, which sticks when the cement hardens, prevents the formation of interstices of excessive size.

Dr. Mason. Dr. William P. Mason.—In the removal of the taste or odor produced by the discharges from gas works, the Albany plant has failed, and in this, any plant must, of necessity, fail. There have been many complaints in Albany in relation to this matter, and the best way out of the difficulty seems to be to arrange with the gas company not to put any of their waste products into the water.

TABLE No. 7.—Comparison of Costs of Operation of Filters at Albany, N. Y., with those in Several European Cities.

Costs are based upon an 8-hour day at $1.50.

Location of Filters.	Items.	Time in man-hours per acre.	Cost.	
			Per million gallons.	Per acre.
Albany.......	Scraping dirty sand into piles.........	61.1	$0.19½	$11.45
	Wheeling out dirty sand and leveling beds..................................	111.0	0.35½	20.81
	Washing sand, wheeling it back to filter and leveling the top: $1.13½ − (0.19½ + 0.35½) = $0.58½	....	0.58½	....
	Total cost of cleaning................	....	1.13½	32.26
Liverpool....	Scraping. (Labor, $0.91 for 10 hours per day)............................	....	....	12.50 to 25.00
London...... (New River.)	Scraping and wheeling out dirty sand, costs $13.39 per acre, with labor at $0.93 for 10 hours, or, based on an 8-hour day at $1.50..............	144.0	....	27.00
London...... (Southwark and Vauxhall.)	Scraping and wheeling sand to sand-washer costs $38.00 per acre, with labor at $0.95 for 10 hours, or, based on an 8-hour day at $1.50............	400.0	....	75.00
Schiedam....	Scraping and wheeling out dirty sand costs $10.44 per acre, with labor at $0.60 for 10 hours, or, based on an 8-hour day at $1.50	174.0	....	32.62
Rotterdam...	The mean cost for filter management during 10 years has been............	....	$1.53	....

The speaker, being interested in the subject, has made some experiments in reference to this flavor or odor. The English plan of filtration was a failure, likewise the mechanical filters; but the speaker has found that, after the water had been treated in a mechanical plant, the difficulty was overcome by passing the water through carbon. The simplest way, however, is to prevent the gas material from entering the water. Dr. Mason.

The figures in Table No. 7, in reference to the operation of filters, have been arranged in such a manner as to facilitate comparison with the cost of operating some of the European filter plants.

CHARLES E. FOWLER, Esq.* (by letter).—The writer has had no experience in the operation of covered filters, and, therefore, cannot be certain that there are not difficulties in their operation unknown to him. Judging, however, from his experience in the use of open filters, it would appear that the usual hindrances to successful operation had been entirely removed in the construction of the Albany plant. Indeed, it would seem that the only difficulty to be apprehended in the operation of the plant would be the tendency to place undue reliance upon the automatic appliances which, however perfect and complete, require watching. To one accustomed to battle with atmospheric conditions in operating an open filter plant in this latitude, the supervision of such a plant as described would seem a pleasant pastime. Mr. Fowler.

It is ordinarily understood that the chief object to be attained in covering a filter is to obviate the difficulties and expense arising from the action of frost, and this possibly may be the chief, but it is by no means the only, object to be sought, at least in dealing with a water having the characteristics of the Hudson River water at Poughkeepsie. The algæ growths on the sand in summer are quite as troublesome and almost as expensive as ice and frost in winter. Like ice, they can develop on an unlimited area in the same time as on a small unit, and will stop a filter and put it out of service just when it should be otherwise doing its best work. Covering, as described in this paper, will prevent the development of algæ entirely. During the summer of 1899 the filters at Poughkeepsie were scraped six times, from May 1st to October 10th, at a cost of about $70 for each scraping. Had it not been for algæ growths, three scrapings in that period would have been sufficient. The cost of the extra scrapings necessitated by the algæ growths, together with the cost of washing and replacing the additional sand removed, amounted in 1899 to more than the cost of removing ice.

There is still another evil incident to an open filter in the hot months. It is essential to the proper working of a filter that, in scraping, as little sand as possible be removed, and that the depth removed

* Superintendent and Engineer of Public Works, Poughkeepsie, N. Y.

Mr. Fowler. be as nearly uniform over the entire area as possible, about ½ in. being the ordinary depth. In the hot months, after a filter has been in use for two or three years, the surface of the sand, when exposed for scraping, and thus subjected to the rays of the sun, will bake, in the course of a few hours, to a depth of 1 or 1½ ins., or even, in extreme cases, 2 ins. It frequently happens, therefore, that in commencing to scrape a bed in the morning the normal thickness will be removed, but before the work is finished, on account of the action of the sun on the exposed surface, it may be necessary to remove three or four times the normal thickness, thus lessening the efficiency of the bed and largely increasing the expense of scraping. The writer, therefore, from his experience, would urge the covering of filters in any climate, particularly for river waters similar to those of the Hudson.

From the writer's experience it is essential that a filter should always be filled, to a short distance above the sand surface, from below. It seems impossible to admit a current of water on the dry sand so as not to wash or furrow it, and even after filling, the escape of the confined air greatly disturbs the uniform compactness. The writer, therefore, notes with pleasure the completeness of the arrangement for filling the Albany filters from below.

The writer is of the opinion that the single filters of the Albany plant have as great an area as can be operated with economy and efficiency. However great the plant to be installed, it should be composed of units no greater in area than those described. Units of lesser area may be used where desired; indeed, it is the writer's opinion that no plant, however small, should be composed of less than four distinct units of equal area, combined as in the plant under consideration.

The quantity of gravel used appears to have been reduced to a minimum. This is rendered possible by the concave form of the bottom. It may, however, be questioned if the depth over the collecting pipes, 4½ ins., is sufficient to insure, for the greatest period practicable, against sand reaching the collectors and obstructing them. When the old filter at Poughkeepsie, designed by the late James P. Kirkwood, M. Am. Soc. C. E., was opened for repairs in 1897, after having been in operation twenty-five years, sand was found all the way down to the bottom of the filter, although the original depth of stones and gravel was 4 ft.

The depth of broken stones was 2 ft. Above the stones were four courses of gravel, each 6 ins. in thickness. In the absence of actual measurement, it is believed that the mean effective size of the gravel in the upper course was but little if any greater than that in the upper course of the Albany filters, though it was evidently less carefully screened. One of the causes tending to force the sand down into the gravel was, undoubtedly, the invariable practice of filling the bed

from the surface,—there being no other means provided. The avoidance of this practice, in the Albany filters, together with the greater care in screening the gravel, will lessen the tendency of the sand to pass downward; nevertheless, the writer believes that some sand will, in time, however remote, reach the collectors, and that an additional thickness of gravel would prolong that time in a ratio greater than that of the increased thickness. It seems to the writer that about double the thickness of each grade of gravel over the top of the conductors would remove the time of possible obstruction of the collectors to a period in the future sufficiently remote to justify the additional expense of construction. Mr. Fowler.

The device for aeration, numerous small holes near the top of the vertical extension of the inlet pipes to the settling basin, would seem to require the addition of some appliance for keeping the holes clear and preventing their becoming closed by algæ and floating or suspended matter.

The devices for preventing undue loss of head or pressure upon the filters, for observing and regulating the rate of filtration, and for preventing an excessive rate, appear to be admirably adapted for their respective purposes.

The writer congratulates the author and all associated with him in the design and construction, as well as the City of Albany, upon the successful installation of a filtering plant apparently so nearly perfect in all its arrangements for successful operation.

GEORGE W. FULLER, Assoc. M. Am. Soc. C. E. (by letter).—This important paper, dealing with the largest plant of its kind now in operation in this country, is a valuable contribution to the subject of water purification. In many ways this plant shows the results of careful thought and of thorough studies of the experimental evidence obtained in this country, and also of the construction and operation of municipal plants in Europe. Mr. G. W. Fuller.

Comparing the Albany plant with those constructed years ago in Europe for the purification of the general type of water of which the Hudson River is representative, it is found that this plant contains many improvements. It is thoroughly modern, embodying the results of the progress of the last dozen years in this particular line. For many years the construction and results of operation of the Albany plant will doubtless be studied by engineers interested in water purification.

In reading this very interesting paper there occurred to the writer a number of points of inquiry and comment, the principal of which are as follows:

Character of Hudson River Water with Reference to Turbidity and Color.—The paper deals at considerable length with the sewage pollution of the unfiltered water, but leaves the reader to his general infor-

Mr. G. W. Fuller. mation with regard to the nature, degree and duration of turbid water which occurs in the Hudson at Albany. Obviously, this factor was of some significance in the design of the plant, for the reason that it was decided to construct a sedimentation basin. All members of this Society are doubtless familiar with the general facts that the Hudson flows through glacial drift formation and not through a clay-bearing region; that the headwaters of the river are in a mountainous country, clad in the winter with snow which melts rapidly in the early spring so as to produce freshets; and that at Albany the river is a comparatively short one. In the near future specific information will presumably be available on this subject; but the writer desires to inquire what the general evidence now available shows as to the amount and character of suspended matters in the water during freshets; and also the frequency, intensity and duration of freshets.

It is stated at the close of the paper that the filtered water is satisfactory in appearance regarding color, although it is not stated how much color is found in either the unfiltered or filtered water. The question of how much color due to dissolved organic matters may be present in a water of satisfactory appearance is one upon which there is much difference of opinion. And it is of especial significance in connection with the type of plant adopted at Albany, because, ordinarily, it is possible by this means to remove only about one-third of the color of the applied water. While satisfactory results were doubtless obtained in this particular instance, this is, nevertheless, an interesting topic for discussion.

Filtering Materials and Underdrains.—In connection with the efficiency of the filter, the filtering materials and underdrains are of prime importance. The Albany plant, in general terms, represents the best modern theory and practice in these particulars, and it is here that this plant differs most from the older ones in Europe. Formerly it was the practice to use gravel layers much thicker than was necessary to support the sand and conduct the filtered water to the collecting pipes, and, in some instances at least, the layers were not properly graded to prevent the upper layers from settling into the lower ones. At the present time many reliable data are available to serve in making these computations. Concerning the use of the thin graded layers of gravel, there is no doubt that they conduce to both efficiency and economy. In the case of the Albany plant they appear to be worked out as finely as it is safe to adopt in practice. In fact, in the opinion of some engineers it might be considered questionable whether, with only three grades of gravel, it is advisable to use with the finer grades such thin layers as 2 to 2.5 ins. The experience of the writer shows that ordinarily no difficulty should arise under these conditions, but that rigid inspection of the work is necessary to guard against the layers settling together. The writer desires to inquire if

any evidences have been detected of the sand passing into and through the gravel layers. Mr. G. W. Fuller.

The practice of placing no gravel layers beneath the sand within a distance of 2 ft. 4 ins. of the walls of the filter is especially commendable. If this had been adopted in the older filters it would have doubtless precluded many instances of unfiltered water reaching the filtered water drains.

Filter Covers.—Although there are a number of open filters in service in the general section of the country in which Albany is located, the question of the advisability of covering these filters is too obvious to be a fitting one for discussion. The principal point, in this connection, is to note that for the first time in this country the vaulting for filter covers has been made entirely of concrete. This departure results in economy, and doubtless will be adopted in many instances in the future.

Pure-Water Reservoir and Control of Rate of Filtration.—The description and discussion of these phases of the plant are among the most interesting parts of the paper. They are unusual and unique in a number of ways, and, while not such as to be a model type under many conditions elsewhere, they appear to serve their purpose admirably in this case and to be based on sound reasoning.

Cost of Filters.—The cost of these covered filters, $45 600 per acre, exclusive of land and engineering, is much less than the general figures obtained from other and earlier plants. While there is no doubt that efficient and durable filters of this type can be built, ordinarily, much more cheaply than has been generally considered to be the case, yet the conditions for construction at Albany were usually favorable in a number of ways, as follows:

1. The filter site was a level tract of land, requiring practically no grading and no excavation other than that necessary to obtain material for the embankments.

2. Very little rock excavation was required.

3. The floor of the filters rests on blue or yellow clay, as compared with the quick-sand and the made land which would be encountered in some localities.

4. The conditions, apparently, were free from complications, and required no very expensive steps relative to the exclusion of groundwater from the plant. Concerning the leakage of the filtered water, in the event of cracks in the fairly light masonry, the ground-water level would cause the loss of water to be very small, compared with conditions found elsewhere.

5. Construction materials were much cheaper at the time the Albany contracts were let, than at present.

6. The site was very favorably located with reference to securing and delivering the various materials of construction.

Mr. G. W. Fuller.

7. With open filters, heavier masonry in some respects would be required than in the case of covered filters.

In noting these points, in regard to which Albany was very fortunate, it is, of course, obvious that they do not detract from the merit of this paper on a plant which, in the writer's opinion, is entitled to great praise.

Mr. Whipple.

GEORGE C. WHIPPLE, Assoc. M. Am. Soc. C. E. (by letter).—The writer wishes to express his appreciation of the service which the Water Department of Albany has rendered to the public in the construction of the filter, so well described by Mr. Hazen. Not only will this filter prove a blessing to the citizens of Albany by the saving of lives, but it will stand as a model for American engineers and an object lesson to certain American cities which have been negligent in protecting themselves from the dangers of their polluted water supplies.

The brief period during which the Albany filter has been in operation is not sufficient to show exactly the degree of purification of the Hudson River water which will be attained permanently, but the figures presented by Mr. Bailey show that already good results are being obtained, and that the bacterial efficiency is improving steadily.

On the other hand, it has been found that there are certain things which the filter will not do. It will not remove all the coloring matter from the water. Experts realize that this is not to be expected, but the ordinary consumer does not understand why the filtered water should not be colorless. Experiments have shown that simple sand filtration is not capable of removing more than about one-half of the coloring matter from water, under favorable conditions, and that ordinarily the amount of reduction is not more than one-third or one-fourth. Mr. Bailey has stated that the color reduction at Albany thus far has been about 40%, the color of the applied water being at times as high as 0.50 or 0.60 on the platinum scale. The removal of color by a new filter is usually greater than by one which has been long in use, and it is probable that this percentage of removal cannot be always maintained. The removal of coloring matter from the Hudson River water by the sand filters at Poughkeepsie is shown by the following figures taken from analyses made at various times by Dr. T. M. Drown:

	Feb., 1891.	Nov., 1891.	Dec., 1891.	Jan., 1892.	Apr., 1894.
Color of applied water	0.233	0.15	0.60	0.38	0.30
Color of filtered water	0.19	0.10	0.65	0.40	0.25
Percentage of reduction of color	17	33	....	....	17

In the spring of 1899, the writer obtained the following results from the Poughkeepsie filter:

Color of Hudson River water	0.32
Color of tap water in Poughkeepsie	0.26
Percentage of removal of color	19

Mr. Whipple.

It has been found at Albany that sand filtration is not always effective in removing certain odors from the applied water. Professor Mason has alluded to the odor imparted to the water of the Back Channel by waste material discharged from the gas-works, and which, not being removed by the filter, caused some complaint from the consumers. This matter of odor, in water which is to be purified, should not be overlooked by engineers when considering filtration works. Inasmuch as sand filtration cannot be depended upon to remove such odors as those observed at Albany, it is essential that the applied water should be freed from them, and it is a case where prevention is easier than cure.

Many American streams receive large amounts of factory refuse, and in such cases filtration alone may not always render the water palatable. The refuse should be purified before it is allowed to enter the streams. Sometimes very large streams become affected with odors from factory wastes. The Schuylkill River at Philadelphia is a case in point. A few years ago the water in certain sections of Philadelphia acquired a disagreeable odor which resembled that of creosote as much as anything. Through the courtesy of John C. Trautwine, Jr., Assoc. Am. Soc. C. E., the writer was given the opportunity to make some observations as to the cause of this odor. Samples of water were collected in various parts of the city and examined physically and microscopically. The character of the odor showed that it was not due to microscopic organisms, and no odor-producing organisms were found in the water. The odor was apparently due to manufacturing waste, and suspicion fell upon certain paper mills a few miles above the city from which large volumes of refuse material were discharged at certain times during the day. Samples of water collected from the river had the same odor as that observed in the city, and near the mill where the refuse material was supposed to be discharged, the odor was very decided. The microscopical examinations of the samples showed the presence of fragments of wood fiber in the river water and in some of the samples of tap water. The samples which contained the largest amounts of wood fiber gave in general the strongest odors. Furthermore, it was the testimony of many individuals that the odor was not of equal intensity throughout the day, this being due, apparently, to intermittent discharges from the mills.

There is one feature of the operation of the Albany filter which will be watched with interest by biologists, namely, the storage of the filtered water in an open reservoir. It has become a well-recognized principle in this country that ground waters cannot be stored in reservoirs exposed to the light without liability to deterioration from troublesome algæ growths; and the question has already presented itself to the minds of some as to whether the water supply at Albany will not some day suffer from this cause. Time alone will tell whether

Mr. Whipple. their fears will be realized. That diatoms and other microscopic organisms will develop to some extent in the stored water is to be expected, but the experience of Lawrence and of Poughkeepsie would indicate that the chances of serious trouble from such growths are not very great. In this connection the writer desires to ask the author whether microscopical examinations of the water in the reservoir have shown as yet any tendency of the microscopic organisms to develop; whether the reservoir into which the filtered water is pumped has been recently cleaned; and, if not, whether there exists at the present time any considerable amount of sediment at the bottom of the reservoir. Experience with the reservoirs of the Brooklyn water supply, where mixed ground water and surface water is stored in open sunlight, has shown that the deposits which form at the bottom of the reservoirs have an important influence on the development of the microscopic organisms.

Allusion has been made to the growth of algæ upon the surface of open filters and to the annoyance which they cause in the operation of the filter. Some interesting studies of the growths of microscopic organisms over the sand have been made recently by Dr. Otto Strohmeyer, of Hamburg, and Dr. Ad. Kemna, of Antwerp. A brief account of their observations, with some additional studies made by the writer, may not be out of place in this discussion.

At Hamburg it has been the custom for a long time whenever a filter-bed was scraped to examine microscopically the surface film over the sand and to record the organisms present. Microscopical examinations of the water of the Elbe River have also been made. Much attention has been given to the careful enumeration of the different species, and, for the most part, the methods of the planktologists have been followed. The observations have shown that there is a regular seasonal succession of organisms which develop on the sand. During the winter the diatoms alone are represented, but certain species sometimes develop in great abundance. It is during the spring and fall, however, that the diatoms attain their maximum growth. The green algæ appear in the spring and increase during the summer. The blue-green algæ are present in large numbers during the late summer, their growth usually continuing until cold weather. Substantially, the same seasonal distribution was observed at Antwerp. It is interesting to note that it corresponds with the seasonal distribution of the microscopic organisms repeatedly observed in various lakes and reservoirs of this country.

At Antwerp similar studies of the surface scums have been systematically made, but somewhat different methods have been followed. The work has been carried on at the Waelham laboratory of the Antwerp Water-Works Company. Chief attention has been given to the dominant forms. The actual numbers of organisms present have not been

recorded, but the results have been expressed in proportionate parts of the total number present, on a scale of ten, as follows: Mr. Whipple.

Coscinodiscus	$4\frac{1}{2}$
Melosira	4
Cyclotella	$1\frac{1}{2}$
	10

On this date (Oct. 7), therefore, the predominant forms were *Coscinodiscus* and *Melosira*. At another time the following genera were present:

Melosira varians	5
Fragilaria capucina	4
Spirogyra	1
	10

In this country similar results have sometimes been expressed in "number of organisms on 1 sq. cm. of sand." The following represents the results of a microscopical examination of the film at the top of an experimental sand filter. The sample was collected in March, after the filter had been in operation nearly two months:

	Number of organisms over 1 sq. cm. of sand. (In standard units.*)
Diatomaceæ:	
Asterionella	278 000
Cymbella	130 000
Diatoma	150 000
Melosira	10 000
Meridion	25 000
Navicula	7 700
Stephanodiscus	6 500
Synedra	1 100 000
Tabellaria	2 390 000
Chlorophyceæ:	
Closterium	1 200
Scenedesmus	800
Protococcus	60 500
Conferva	12 000
Spirogyra	5 500
Cyanophyceæ:	
Chroöcoccus	5 300
Oscillaria	84 000
Protozoa:	
Trachelomonas	16 000
Ciliata	5 000
Peridinium	4 000
Tintinnus	14 000
Mallomonas	800
Synura	6 000
Codonella	400
Rotifera:	
Anuræa	800
Polyarthra	1 000
Synchæta	8 000
Total organisms	4 324 500
Amorphous matter	2 300 000

* One standard unit equals 400 square microns.

Mr. Whipple. The organisms which develop over the surface of a sand filter may be grouped, for practical purposes, into three classes: those which form a matting upon the sand; those which are attached to the sand but extend upward in filaments or sheets; and those which are free-floating in the water. Perhaps it would be better to say that the organisms are found in these three conditions, because the same organism is sometimes found now on the sand and now above it.

The effects of these three groups of organisms upon the operation of the filter are not the same. The most important effect is that produced by those organisms which form a matting upon the sand. The diatoms and the unicellular algæ are here chiefly concerned. By their growth they form a more or less gelatinous film upon the surface, and as this film becomes denser, the rate of filtration is retarded until finally it becomes necessary to scrape the filter. The algæ which grow erect upon the sand do not thus clog the filter. On the contrary, they prevent clogging to some extent. Their waving, interlaced threads act as a sort of preliminary strainer, removing from the applied water some of the suspended matter which would otherwise collect on the sand. This action continues as long as the plants are in good condition and as long as the evolution of gas is sufficient to cause flotation. When they begin to decay or when they become overloaded with foreign matter they settle to the bottom and help to clog the filter. Kenma found that at Antwerp *Hydrodictyon* was the most effective organism in this process of preliminary straining. The free-floating forms have little influence on the rate of filtration as long as they remain in suspension, although, to some extent, they too play a part in the preliminary clarifying process. But ultimately most of them reach the surface of the sand and help to clog the filter.

Not only do the algæ growths over a sand filter affect the rate of filtration and the frequency of scraping, thereby increasing the cost of filtration; but they exercise an important influence upon the efficiency of the filter. It sometimes happens that the growth of the algæ is so vigorous and the evolution of gas so abundant that great masses of the organisms rise in the water, carrying with them patches of the surface film and leaving bald spots on the sand through which the water passes at too high a rate, with consequent loss of bacterial efficiency. At Antwerp the filter attendants watch for this phenomenon, and reduce the rate of filtration if necessary. It seems probable, also, that decomposition of the organisms at the surface affects the filtered water unfavorably.

During the course of the year the character of the flora changes. This change is often gradual, but at times is very rapid. Kemna has noticed that at the time when certain organisms are rapidly disappearing from the sand the efficiency of filtration is impaired. He attributes this to the changed condition of the surface film caused by

the decomposition of the organisms, but suggests that changes in the bacterial flora may also play an important part. In a recent publication* Dr. Ad. Kemna cites the following interesting experience with *Anabœna*: Mr. Whipple.

During the hot weather of July, 1899, *Anabœna* became abundant over some of the Antwerp filter beds. Knowing the character of this organism and its tendency to impart an odor to the water, he kept a careful watch of the filters, collecting samples of the filtered water twice a day and testing them as to their odor and the amount of ammonia they contained. As long as the *Anabœna* remained in a living condition in the water over the sand, the filtered water was satisfactory, but when the organisms disappeared, on the advent of cold weather, the filtered water acquired a bad taste and the amount of ammonia increased.

The studies made at Hamburg and at Antwerp show, with apparent conclusiveness, that when the vegetation over a sand filter is in a living condition, it is a positive aid to the efficiency of filtration, though it increases the cost of operation. Most of the microscopic organisms have a coating which is somewhat gelatinous, and in many cases the gelatinous material is very abundant. The diatoms and other organisms which grow directly on the sand aid in the formation of the surface film on which the efficiency of filtration largely, but not solely, depends. This fact has been understood for many years. The surface film forms through bacterial agency on covered filters as well as on open filters, but on the latter its formation is assisted by the microscopic organisms.

TABLE No. 8.—THE INFLUENCE OF GROWING *ENTEROMORPHA INTESTINALIS* UPON WATER BACTERIA. (Direct Sunlight.)

Date.	Hour.	Temperature.	Number of Bacteria per Cubic Centimeter.	
			Culture of *Enteromorpha.*	Water without *Enteromorpha.*
July 6th	9 P. M.	18° Cent.	123	122
" 7th	5 A. M.	18° "	156	164
" 7th	9 A. M.	18.5° "	98	210
" 7th	3 P. M.	22° "	5	532
" 7th	7 P. M.	24° "	0	987
" 7th	9 P. M.	21° "	0	1 230

The experiments of Strohmeyer, at the laboratory of the Hamburg Water-Works, have shown that some algæ exercise a sterilizing influence upon the water in which they develop. *Enteromorpha*, for example, is a green alga which is often very abundant on the filter

*"Les Eaux Potables, Extrait de la Belgique Médicale."

Mr. Whipple. beds during the summer. Strohmeyer selected and carefully cleaned some of the young, growing filaments of this organism and put them in flasks which contained about 300 c. c. of water, adding also 1.5 c. c. of a sterile culture solution. Other flasks of water were similarly prepared, but without the algæ. Pairs of these flasks, with and without the algæ, were subjected to similar conditions of light and temperature, and their bacterial contents determined at intervals of a few hours for several days. The result of one of these experiments is given in Table No. 8.

The figures show that at the beginning of the experiment the bacterial contents of the two flasks were practically the same. After standing for eight hours in the dark, the bacteria increased slightly in both flasks. After fourteen hours' exposure to direct sunlight, the water in the flask which contained the growing algæ was practically sterile, while in the other flask the development of the bacteria continued uninterruptedly. A similar experiment, carried on for a longer period in diffused light, gave the results shown in Table No. 9.

TABLE No. 9.—THE EFFECT OF GROWING *ENTEROMORPHA INTESTINALIS* UPON WATER BACTERIA. (Diffused Light.)

Date.	Hour.	Temperature.	Number of Bacteria per Cubic Centimeter.	
			Culture of *Enteromorpha.*	Water without *Enteromorpha.*
July 4th	11.30 A. M.	18° Cent.	145	108
" 4th	2 P. M.	18° "	160	144
" 4th	6 P. M.	18° "	152	243
" 5th	8.30 A. M.	17° "	1 100	5 900
" 5th	2 P. M.	18° "	180	26 000
" 5th	6.30 P. M.	18° "	7	50 000
" 6th	9 A. M.	18° "	24	63 000
" 6th	7.30 P. M.	18.5° "	0	80 000

Here the development of bacteria in the flask without the algæ was more vigorous than before, but the sterilizing action was less vigorous, as might be expected. Other experiments with *Spirogyra*, *Cladophora* and *Stichococcus* gave similar results. Whether the sterilizing action produced by the growth of the algæ was due to the effect of the gases liberated or to some other cause was not definitely determined.

What is true of these filamentous algæ is probably true of many other microscopic organisms, and, not only of the green algæ, but of the diatoms and, perhaps, the blue-green algæ. At the time when *Anabæna* was present on the Antwerp filters, Kemna observed that it was not equally abundant on all the beds, and that with the beds

where it was most abundant there was not only less clogging of the sand, but an increased degree of purification. Mr. Whipple.

The writer has had frequent occasion to observe the reduction of the number of bacteria in water by growths of microscopic organisms. The following case is very striking: A pond, with an area of 40 acres and a capacity of 42 000 000 galls., was affected with an immense growth of *Clathrocystis* which appeared in the spring and continued until the late autumn. At times the water contained 20 000 standard units of these organisms per cubic centimeter. The water that entered the pond was polluted, and the number of bacteria in it was seldom lower than 1 000 per cubic centimeter, and was often much higher. At the same time the number of bacteria in the water at the outlet of the pond was very low. On one occasion the following results were obtained, and these are typical of the conditions which prevailed for several months:

	Number of Bacteria per cubic centimeter.
Average of the influent streams	1 400
Average of samples throughout the pond	27
Sample at the outlet	36

A series of examinations of samples collected at the outlet of the pond during an entire year gave the following results:

	Number of bacteria per cubic centimeter.	Number of standard Units of *Clathrocystis* per cubic centimeter.
January	.....	120
February	1 100	800
March	1 500	0
April	338	680
May	370	2 320
June	139	14 500
July	60	17 200
August	104	11 700
September	245	8 750
October	540	8 970
November	840	1 150
December	1 243	650

These figures show that the number of bacteria at the outlet of the pond varied inversely with the amount of *Clathrocystis* present. In this case a part of the reduction of the bacteria, at least, appeared to be due to a mechanical action by which the bacteria were engulfed in the mass of jelly in which the cells of *Clathrocystis* were embedded. Microscopical examination of the *Clathrocystis* showed that various kinds of bacteria were present in this gelatinous mass—micrococci,

Mr. Whipple. long bacilli, short bacilli, singly and in groups. The bacteria thus absorbed may or may not have been in a living condition, but at least they were incapable of developing on the gelatin plate. Microscopical examination of the cultures in the Petri dishes showed that in very few instances was a mass of *Clathrocystis* the nucleus of a colony of bacteria. Laboratory experiments were equally suggestive of the action of *Clathrocystis* on bacteria. A water which contained 430 bacteria per cubic centimeter was mixed with an equal volume of a water very rich in *Clathrocystis*, which contained but 39 bacteria per cubic centimeter, and shaken vigorously. Plate cultures of the mixed waters resulted in the development of only 60 bacteria per cubic centimeter. Other experiments of a similar character corroborated these results.

In the reservoirs of the Brooklyn Water-Works, it has been observed repeatedly that when *Asterionella*, *Synedra* and other organisms have been present in great abundance, the numbers of bacteria have been unusually low. There is some reason to believe that the microscopic animal growths tend to reduce the number of bacteria in water by consuming them as food. The relation between the water bacteria and the lower forms of microscopic plants and animals is certainly an intimate one, and the subject is one which deserves careful study, as it is not only important in connection with filtration, but has a direct bearing upon the self-purification of streams and other natural phenomena.

Mr. Rafter. GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—This paper presents, in an interesting manner, the detail of a continuous filtration plant constructed substantially on lines followed for many years in England, and on the Continent of Europe generally. The cost of the plant per unit area is lower than that of the European plants, which is attributed to the favorable conditions at Albany. Basing his view on perhaps casual study of filtration plants abroad, the writer is, however, disposed to say that the decrease in cost is partly due, at any rate, to somewhat less elaborate construction than is common there. The extensive use of concrete has also contributed to keep the cost within moderate limits.

The author has referred to studies of the flow of the Upper Hudson, made by the writer. The cited figure of 1 657 cu. ft. per second, minimum flow, is probably not far from right, although in the summer of 1889, for a few days, the flow may have been as low as 1 200 to 1 300 cu. ft. per second.

Table No. 10 gives the run-off of the Hudson River in cubic feet per square mile per second, for a period of 12 years, from 1888 to 1899, inclusive. These figures do not represent the natural flow of the spring months, which are modified by a lumberman's storage of from 3 000 000 000 to 4 000 000 000 cu. ft. The Indian Lake Reservoir, of

4 470 000 000 cu. ft. content, was also brought into use in 1899, the storage therefrom being discharged into the stream during July, August and September. Had it not been for this large artificial inflow, the minimum of these months would have been considerably lower than shown. Mr. Rafter.

TABLE No. 10.—Run-Off of Hudson River at Mechanicville, for the Water Years, 1888 to 1899, Inclusive, in Cubic Feet per Second per Square Mile of Catchment Area.

(Catchment Area = 4 500 Square Miles).

Period.		1888	1889	1890	1891	1892	1893	1894	1895	1896	1897	1898	1899
Storage.....	December...........	1.78	2.22	2.93	0.72	1.91	0.90	1.60	0.97	2.42	1.54	3.05	1.22
	January............	1.41	2.44	2.50	1.84	4.19	0.71	1.50	0.86	1.51	0.89	1.72	1.49
	February	0.82	0.84	1.76	2.58	2.06	1.07	1.08	0.79	1.04	0.87	1.50	1.17
	March..............	1.52	1.84	2.47	3.94	2.43	1.83	3.28	0.93	3.02	2.49	4.49	2.14
	April..............	4.73	3.04	3.34	4.44	4.79	3.98	2.47	5.31	5.55	4.24	3.07	5.25
	May................	4.76	1.97	4.00	1.22	4.36	4.95	1.68	1.52	1.02	2.70	2.47	2.17
	Means	2.55	2.08	2.85	2.45	3.30	2.24	1.96	1.72	2.42	2.12	2.72	2.23
Growing	June	1.09	1.52	1.64	0.71	2.76	1.07	1.58	0.63	1.05	2.64	1.17	0.58
	July	0.34	1.28	0.43	0.52	2.08	0.56	0.70	0.50	0.62	2.39	0.57	0.54
	August.............	0.38	0.95	0.45	0.59	1.22	1.11	0.55	0.87	0.54	1.83	1.13	0.31
	Means	0.60	1.21	0.83	0.65	2.00	0.91	0.94	0.67	0.74	2.29	0.96	0.47
Replenish-ing.	September..........	0.63	0.44	1.96	0.45	0.99	1.53	0.41	0.58	0.51	0.51	0.84	0.46
	October............	1.02	0.83	2.04	0.33	0.63	0.86	0.81	0.58	0.91	0.56	1.75	0.58
	November...........	2.36	1.77	2.03	0.91	1.69	0.81	1.42	1.87	2.97	2.21	2.05	1.42
	Means	1.34	1.00	2.02	0.56	1.09	1.06	0.88	1.01	1.46	1.09	1.55	0.82
Water Year...	Means	1.74	1.60	2.13	1.53	2.42	1.62	1.43	1.28	1.74	1.93	1.98	1.44

In his statements relating to cracks in the walls, the author apparently assumes that cracks in masonry are, on the whole, to be expected. This has always seemed to the writer so far an erroneous view that he is disposed to say that even in our severe winter climates, cracks in concrete masonry ought not to occur. At any rate, this general statement may be made for walls of any considerable thickness. For thin walls, the statement may possibly be modified somewhat, although there are certainly cases of such, of considerable length, free of cracks.

The writer is disposed to assign lack of cleanliness as a prolific source of so-called temperature cracks. As a general statement, for walls as long as those referred to by the author, the elasticity of the mortar ought to take care of expansion and contraction. But this implies thorough adhesion of the mortar. The chief remedy is, of course, cleanliness of stone and mortar materials.

By way of showing the elasticity of concrete under compression, reference may be made to the writer's studies of concrete as recorded in the Annual Report of the State Engineer and Surveyor of New

Mr. Rafter. York, for 1897, where tabulations of a large number of compression tests may be found. Without going into an extensive *résumé* of these tests, at this time, a few points may be cited in illustration of the writer's proposition.

Table No. 11 gives the serial number as per report to State Engineer, brand of cement, resilience and modulus of elasticity for a number of concrete blocks of 1 to 3 plastic mortar, 40% of the aggregate. The resilience here tabulated is for a gauged length of 5 ins., and a compression of 600 lbs. per square inch, while the modulus of elasticity is between loads of 100 and 600 lbs. per square inch.

TABLE No. 11.

Serial number.	Brand of cement.	Resilience for length of 5 ins.	Modulus of elasticity.
23	Genesee	0.0020	1 250 000
51	Wayland	0.0014	1 786 000
72	Iron Clad	0.0018	1 923 000
90	Empire	0.0011	2 273 000
109	Champion	0.0016	1 562 000
	Mean =	0.0015	....

For a length of 1 ft., and with 600 lbs. per square inch compression, the mean resilience of the foregoing blocks is therefore found to be 0.0036 in. At this rate a concrete wall of the specified composition might be expected—if free to move—to expand nearly 1½ ins. before serious consequences would result. Taking into account that so-called temperature cracks do not often exceed, even in walls several hundred feet in length, from ⅛ to ¼ in., it has seemed probable to the writer that, with the proper precautions taken, they ought not to occur. At the same time, there is no intention of being specially insistent on these points, but rather to present them in the hope of eliciting further and, possibly, more profitable discussion.

For ordinary brick walls, the writer recognizes that they are very liable to crack under some of the conditions detailed in this paper.

Tables Nos. 11 to 16, inclusive, of the writer's report on Concrete, in the Annual Report of the State Engineer and Surveyor of New York, for 1897, give results of a large number of compression tests and computed moduli of elasticity for concrete and mortar. Additional data of this character may be found in the "Report on Tests of Metals and Other Materials, Watertown Arsenal, 1898." Indeed, recent compression tests have so multiplied as to enable one to form a fair idea of the elasticity of concrete and mortar. For other masonry materials there is still a great lack of data.

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PAPERS AND DISCUSSIONS.

THE REACTION BREAKWATER AS APPLIED TO THE IMPROVEMENT OF OCEAN BARS.

Discussion.*

By Messrs. E. L. CORTHELL, A. F. WROTNOWSKI and LEWIS M. HAUPT.

E. L. CORTHELL, M. Am. Soc. C. E. (by letter).—The last paragraph of the paper refers to the channel at the mouth of the Brazos River, Texas, in which it is stated that, contrary to the expectations of increased depths by the great flood of July last, the sounding showed but 15 ft. in several places, "thus demonstrating the relative superiority of the principle of reaction by a single jetty." Mr. Corthell.

From a newspaper clipping it is learned that the United States Assistant Engineer in charge of the surveying expedition at the mouth of the Brazos River, has reported to Captain Riche, U. S. Engineer, in charge of the Brazos District, that there is a depth of 18 ft. of water at mean low tide at the mouth of the river, and that this depth extends not only from the Gulf through the jetties, but to 500 ft. above the lighthouse, which is some distance inland. The works at the mouth of the Brazos River were built in reference to a datum plane of average flood tides; this would make the depth at mean high tide over 19.8 ft. A more recent survey by the Government Engineers shows that there is, throughout the entire length of the jetty channel, a

* Professor Haupt's paper on this subject, together with all the discussion thereon which had been received up to December 31st, 1899, has been published in Volume xlii of *Transactions*. Subsequently, the discussion in the *Proceedings* for January, 1900, and the present discussion have been published. Additional discussion on this subject will be collated and published in subsequent numbers of *Proceedings* and in the next volume of *Transactions*.

Mr. Corthell. minimum depth of 20 ft. at mean low water, 21.8 ft. at mean high water, the datum plane of the works.

The feature to which attention is particularly called by the writer is, that although the high water came in July last, and the channel at that time was deepened to 18 ft. at mean low water, it has maintained itself through the seven months intervening between that date and the date of the survey by the United States Engineer.

Mr. Wrotnowski. A. F. Wrotnowski, M. Am. Soc. C. E. (by letter).—While the conditions at Aransas Pass, resulting from the construction of its disconnected reverse-curve breakwater, seem to have fulfilled in a measure the expectations of the Board of Engineers who planned it, much remains to be done before it can be accepted as a success in the improvement of tidal harbors.

The very essence of the jetty principle of concentration of the volume of water, whether it be tidal or fluvial, between parallel dykes, is at stake, in this so-to-speak experiment; and should it prove efficacious in all its features, the great reduction in cost alone would certainly recommend its adoption.

There is no doubt in the writer's mind that the curved trend of a breakwater or jetty adds materially to the forces for scour of the channel, over that of a straight jetty. The tendency of a moving body of water in a confined channel with an uneven and rough bottom will always cause eddies, and a crooked channel, more or less tending to cross and recross from side to side, making an irregular cross-section; but with a properly and regularly curved jetty the tendency of the volume of water is to "hug" the concave side of the channel.

The sectional area in a curve or bend of a stream, be it between jetties or in a natural water-way, is always greater than in reaches. This will always be the case in the whole length of the segment of the curve, and this applies equally to a channel in an open way, especially when it is guarded in its trend by a breakwater, as is the case at Aransas Pass.

The system of two tidal entrances or debouchures, as in this case, is especially applicable where tides are slight, as they are in the Gulf of Mexico, because, on account of the comparatively small tidal volume to be depended upon for scour, it is essential to secure all the volume possible to cause the required scour, and so this system may well be tried in such small tidal localities.

But at other points, for instance, on the Pacific, along the United States and Mexican coast, where the shores are very abrupt and defiant, and the tides rise much higher, it is doubtful if the system could be applied. For the port of Altata, State of Sinaloa, Mexico, the writer has had occasion to propose a single jetty for the maintenance of a given depth at the new entrance to this port, which was breached through the Peninsula about 10 miles south of Altata during the hurricane of

November, 1896. The writer had then in contemplation a detached jetty, but after having made the surveys he found the conditions not adapted, principally on account of very unstable and deep shores. He found, also, that the volume passing in and out was sufficient to accomplish the required scour and keep a permanent depth; therefore, a single curved jetty was proposed. The Government, however, was not ready to expend the required funds for its construction until a better financial condition was apparent. Mr. Wrotnowski.

The auxiliary tidal mouth in the Aransas Pass work is open to question as to whether it is an advantage. During high wave-energy, there will likely be brought into the channel quantities of sediment which will deposit to the lee, and which, during storms of long duration, will accumulate in such quantities as to cause much delay afterward in clearing the channel of such deposition, by the disturbed forces operating in the channel.

The writer has seen considerable such deposition through breaches, especially at Tampico and Vera Cruz; in one case, at the last-named place, where it filled a radial space of about 300 ft. to a depth of 20 ft., and it covered entirely the inside apron of the breakwater for over 500 ft. Not only did it deposit, but it kept on accumulating in the harbor for a time, causing excessive expense in dredging, until the breaches were closed. It is true, however, that in this case there was not sufficient tidal energy inside the harbor to carry away such deposit.

At Tampico the case was different, and the deposition which took place there was readily swept away, the conditions therefor being favorable.

Lewis M. Haupt, M. Am. Soc. C. E. (by letter).—Mr. Wisner cites an ideal case for the use of such a breakwater, but, unfortunately, it was in consequence of the effort to apply this ideal by "continuing the natural curve of the outlet" that caused the work of the Government to fail at Aransas, for the reason that the "natural curve" is the result of the external drift encroaching upon the advancing or convex side of the inlet, and is therefore on the far or wrong side of the channel. A jetty built on the concave side, therefore, deposits the littoral drift in and blocks up the channel, causing the bar to move more rapidly seaward. Hence, it is not generally good practice to continue the "natural curve" without an auxiliary structure abreast of it to arrest the drift, thus requiring two jetties if the far one is built first. Mr. Haupt.

As several members appear to be under the impression that the single curved breakwater is not applicable to the mouths of sediment-bearing rivers, the writer desires to state his reasons for entertaining a different opinion. The objection is based upon the statement "that any diminution of velocity of flow produces deposition."

This is conceded as correct on general principles, but subject to

Mr. Haupt. the modifications as to effects of velocity in producing scour, as noted by the writer and confirmed by Professor Williams, and Mr. Wisner in his former papers. Hence, the only question arising is as to which form of construction produces the greater diminution of velocity, one curved jetty or two straight ones; and are their effects upon the outflowing currents at all similar?

The writer's position as to two rigid jetties which are practically parallel is, as already stated, that they act merely as an aqueduct to convey the effluent water and its load of sediment over the site of an existing bar merely to be deposited in the sea beyond, where a new obstruction will be formed, unless there is a strong littoral current to prevent it. A single reaction jetty, on the contrary, produces a lateral movement of sediment, thus removing it from the concave side where the velocity is the greatest, because of the longer path, and notwithstanding the lesser slope, to the convex side, where it accumulates and forms a natural levee, which, in process of time, automatically adjusts itself to variations of the regimen of the stream. It thus happens that instead of all the material being carried or rolled to the mouth, as with two jetties, by far the greater portion of it is thrust aside before reaching the outer end of the concave jetty and is deposited outside of the navigable channel, and thus the growth of the bar seaward is greatly retarded. The result is that the one jetty not only costs far less to build, but is also much cheaper to maintain.

These views appear to be sustained by the facts stated by Mr. Wisner, relative to the maintenance of the Eads Jetties at the South Pass of the Mississippi River, authorized in 1875, at an estimated cost of $5 225 000. These jetties were parallel and 1 000 ft. apart—too far, in fact, for the best results from the small portion of the discharge traversing that effluent—but being rigid, the readjustment could only be made by groins and inner jetties which contracted the channel to about 600 ft., thus increasing the cost, and at the same time producing alternations of velocity instead of uniformity, but resulting finally in securing the 30 ft. of depth desired.

Mr. Wisner, however, makes an important statement, viz.: "That the curvature of the channel was such that excessive depths developed near the concave jetty and caused sufficient deposit on the convex side to reduce the width at a depth of 26 ft. to less than 200 ft.," thus showing that the degree of curvature was excessive, since the reaction was too great for the contracted channel; also, that there was a resultant movement of material from the concave to the convex bank. The channel through the South Pass has several reversals of curvature and consequent "crossings," yet the depths are maintained. The east or concave jetty was located as a transition curve starting from a tangent, with a gradually increasing deflection until it reached a radius of 11 720 ft.; its total length being 12 100 ft., nearly 2¼ miles.

The west jetty was parallel to the other for nearly 3 000 ft., and thence to sea it curved with a radius of 15 000 ft. Mr. Haupt.

Mr. Wisner states a fundamental principle when he says: "The amount of curvature given a channel fixes the width that can be maintained." Hence, the success or failure of the engineer depends upon a proper adjustment of his radii to the local conditions; if too short, the channel will be too deep and narrow, and *vice versa*. The happy medium which is best adapted to all stages must be determined.

Hence, the writer is of the opinion that a single curved jetty, adjusted to the regimen of the stream, and having no restraining counter jetty to interfere with the natural deposition of material beyond the normal section of the channel, will not only cost less, but will give a far more satisfactory result as to maintenance.

In support of this view the writer submits a few brief extracts from well-known foreign authorities illustrating the action of curved channels. In his "Tidal Rivers" Mr. Wheeler says:

"A concave bank sets up a scouring action in the current by diverting the particles of the water from their straight course, causing that rotary motion and boring action which occurs in all bends, and which operates in deepening the channel along the concave side."

Mr. Stevenson says: "That it might safely be affirmed that a stream is most likely to follow a permanent course when directed by a concave bank."

Mr. Scott-Russell: "Where the curves were gentle, the natural bends should not be interfered with; that a river has an oscillating motion * * * the mere fact of the commencement of curvature would give it a tendency to continue that curvature, and the stream would go on oscillating regularly to the sea in curves of opposite curvature. Continuity of a system of oscillation should therefore be maintained."

Captain Culver says "that straight reaches are strictly to be avoided * * * since the deep-water track is acted upon by the most trifling causes, ranging from side to side at will * * * there is therefore no security whatever for the permanency of the deep water, either in a fixed channel or at the shipping berths."

The French Government Commission, in reporting on the improvement of the estuary of the Seine, advised that the training walls should be extended in a sinuous form, having a concave bend leading to the entrance to Honfleur Harbor, in order that deep water should be maintained at the entrance.

M. Fontain, in the rectification of the Rhine, avoided straight cuts and adopted curves.

From which it appears that the most experienced authorities recognize the value of curvature as a means of creating and preserving a channel in silt bearing, tidal streams and estuaries.

From all of which it would appear that for tidal estuaries or inlets

Mr. Haupt. on sandy coasts, where drift and wave bars obstruct the entrance, a single reaction breakwater should be placed to leeward of the proposed channel; while for sedimentary or delta rivers emptying into comparatively tideless seas, a single concave jetty, of long radius adjusted to the discharge, will best serve to create and maintain the best channel at least cost.

The cases are widely different, in consequence of the origin and direction of movements of the silt; the one being littoral and external, the other fluvial and internal; hence they require distinctively different treatments.

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PAPERS AND DISCUSSIONS.

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RIVER HYDRAULICS.

Discussion.*

By GEORGE W. RAFTER, M. Am. Soc. C. E.

GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—This paper is an admirable illustration of the present tendency in hydraulics, not to attempt to express complex relations drawn from a large number of cases by a single formula, but rather to work out each case by itself, on its merits. The recognition that the discharge of a stream for a given gauge height will be different for rising from what it is for falling stage, with each in some proportion to rise and fall, is a case in point, as is also the discussion of the effect of "bed in train." Frequently, the tendency has been either to include all such phenomena in a single expression, or to ignore them entirely. The best illustration, however, is found in the conclusion of the paper, that there is a particular equation which expresses the hydraulic relations of rivers better than the formulas in common use, but which does not in any degree apply to pipes, conduits and uniform reaches of straight channel. Mr. Rafter.

While the paper thus illustrates a desirable improvement in hydraulic studies, it contains, further, a series of generalizations which assist one materially in comprehending the complex series of physical facts entering into the flow of a large stream, where bends, irregular bed and other disturbing influences tend to complicate the phenomena. For all such, notwithstanding current practice, it is well to recognize

* Continued from January, 1900, *Proceedings*. See October, 1899, *Proceedings* for Paper, by James A. Seddon, M. Am. Soc. C. E., on this subject.

Mr. Rafter. that $v = c\sqrt{r s}$ can have at best only casual application, and as a demonstration of this point Mr. Seddon's paper can hardly be excelled. But for straight reaches of artifical channel with uniform cross-section, the conditions of flow are so different from those of meandering and silt-bearing streams, that deductions applicable to one may not apply in any degree to the other. For such a channel, the theory of velocity-slope relations becomes, the same as for pipes and conduits, all important. As shown by the author, the views expressed in the paper do not apply to these cases, but are to be considered as confined to large streams with relatively flat slopes. Nor, so far as the writer can now see, will they apply to small streams and mountain torrents, for both of which the Chezy formula is more nearly applicable. This, however, is merely in line with the proposition to, so far as possible, work out formulas suited to each specific case.

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THE SOUTH TERMINAL STATION, BOSTON, MASS.

Discussion.*

By Messrs. HERMAN CONROW and J. R. WORCESTER.

HERMAN CONROW, Jun. Am. Soc. C. E. (by letter).—The method used by Mr. Francis in water-proofing the masonry throws new light on a subject upon which comparatively little has been written. Mr. Conrow.

In the summer of 1896 the writer had charge of the construction of a water-proof subway, beneath the four tracks of a railroad in Massachusetts. The subway was about 100 ft. long and 8 ft. in clear width, with the floor about 5 ft. below the level of the water in a nearby creek. It was necessary, therefore, to build a masonry structure which would be impervious to water under pressure. The design of the subway called for a foundation of concrete, brick side and end-walls, and a brick-arch roof with concrete backing. The water-proofing materials were refined bitumen asphalt of the best quality and heavy tarred paper. Inside, the finished subway was planned to have a mosaic floor and sides and arched ceiling of glazed bricks; but these embellishments have nothing to do with the water-proofing, and were added merely as a veneer to the main walls of the structure. The subway was built in a soil of fine sand so full of water that it was not far removed from a quicksand.

* This discussion (of the paper by George B. Francis, M. Am. Soc. C. E., printed in *Proceedings* for December, 1899), is printed in *Proceedings* in order that the views expressed may be brought before all Members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to March 30th, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Conrow. In order to simplify the description, the methods used in water-proofing the floor, side-walls and roof are separated.

1. The floor was water-proofed as follows: A foundation course of concrete, 6 ins. thick, was laid upon the sand, and the top of this was roughly leveled and smoothed by filling the depressions with mortar. After this mortar had set, heavy tarred paper was laid to break joints, and over the paper was poured a layer of molten asphalt. (The thickness of this pouring will depend upon the temperature of the asphalt. Very hot asphalt will run out to a thickness of less than ¼ in.; but it can be used so that the pourings will be nearly ½ in. thick. The thin pourings, however, give the best results.) On this asphalt, after it had cooled, another layer of tarred paper was placed, which was followed by another coating of asphalt. This process was continued until a water-proof covering about 1 in. thick was formed. This was composed of three layers of paper and three pourings of asphalt. This floor covering could have been made entirely of asphalt, but the tarred paper greatly increased its strength.

Upon this water-proof floor 6 ins. of concrete were next laid, except where a space was left for the asphalt in the vertical walls to join directly to the asphalt floor. The purpose of this concrete was to keep the asphalt permanently cool and also to withstand the water pressure from beneath. Steel rails, 2 ft. apart, were placed in the concrete, and the ends of the rails were bedded under the inside walls, thus giving strength to the concrete, in addition to its weight, to resist the upward pressure, otherwise a heavier layer of concrete would have been necessary.

2. To water-proof the side-walls of the subway the general scheme was to build a core-wall of asphalt, 2 ins. thick, between two brick walls. The greatest care was necessary, in making the junction at the bottom, where the asphalt core-wall met the asphalt floor, to secure a water-tight joint. The inside wall surrounding the core was first built to a height of 6 ins. and the outside wall to a height of 1 ft., thus forming a narrow groove 12 ins. deep, into which molten asphalt was poured. In order to reduce the amount of asphalt, broken pieces of clean and dry brick were laid in the groove, which was then grouted full with very hot asphalt. The core-wall having thus been raised level with the brick walls about it, the brick walls were built up 12 ins. higher, and the process of asphalting repeated. In this manner the sides and ends of the subway were built to the desired height. When this had been done the walls and bottom formed a water-tight masonry box.

When starting the core-wall at the bottom, the asphalt was poured very slowly, for there was danger that the hot asphalt would melt that under the concrete and cause it to crack; but after the core-wall had reached a height of 1 or 2 ft., all anxiety on that score was dismissed.

To make a tight junction between one pouring of asphalt and another, the surface of the cold asphalt must be perfectly clean and dry. A film, either of dust or moisture, will prevent a water-tight junction. Sometimes, when the brick walls had been freshly built up above the core-wall, particles of mortar and drops of water collected upon the surface of the cold asphalt in the groove, and, in spite of brushing and wiping, it could not be cleaned and dried perfectly. In such cases a small quantity of kerosene was sprinkled on the surface of the asphalt in the groove and then lighted. The heat dried out the moisture and melted the surface of the asphalt, giving a perfectly new surface and making sure the unity of the work. Mr. Conrow.

This use of kerosene the writer has found very valuable. Old surfaces of asphalt which had become full indentations and which were covered with sand which had become ground in, were rendered bright and clean by burning kerosene on the surface; the foreign particles disappeared by sinking deeper into the asphalt, leaving the surface in good condition to unite with the next pouring.

3. The water-proofing of the arch covering was very simple. After the masonry had been smoothed over with cement and it had set, asphalt was poured evenly over the surface, and, after cooling, a layer of tarred paper was laid, beginning at the sides and parallel with the barrel of the arch, working upward toward the crown, each course lapping over the previous one as in a shingle roof. Four layers of asphalt and three of paper were used.

The water-proofing of the subway by these methods was entirely successful, and upon the completion of the work the pumps were stopped and the water rose to a height of 4 ft. around the walls of the subway, which remained perfectly dry inside. Numerous heavy rains caused no leakage in the arched roof.

This method of water-proofing the side walls was not that originally tried upon this work. At first the attempt was made to water-proof a brick wall by means of paper and asphalt. After the wall had become dry it was coated with asphalt, and then paper was put on with lap joints. Another coat of asphalt followed, then more paper, and so on. A 4-in. brick wall was built adjacent to the paper to hold it and the asphalt in place. The results were not successful, and perfect water-proofing was not secured in this way. The writer is of the opinion that tarred paper has little value as a water-proofing material for vertical walls which are to stand water under pressure, because of the impossibility of keeping the paper in its proper place. He also believes that if it is desired to build masonry absolutely water-tight the best way to do it is by using an asphalt core-wall where the asphalt may be poured into a groove and allowed to run into every crevice which will admit water.

Both the asphalt and the tarred paper should be of the best quality.

Mr. Conrow. Cheap asphalt and the tar compound are not permanent substances, but will rot and become like powder under the action of water and air, thereby loosing all water-proofing qualities; and the poorer kinds of tarred paper are easily torn and are of little value. It will be found very difficult to handle the asphalt in hot weather since it becomes soft in ordinary summer temperatures. Cold asphalt is a durable and tough substance, capable of bearing considerable pressure, but if unprotected from the sun's rays, it becomes an unreliable material. If it is

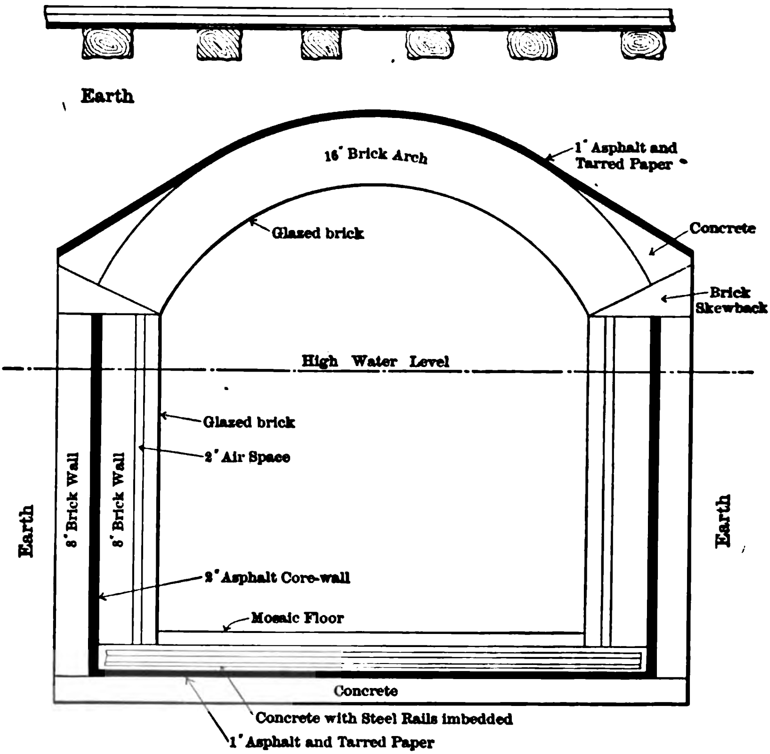

Fig. 19.

desired to use a large quantity of asphalt, the inconvenience of melting it quickly may become very great, especially if the asphalting outfit consists of but one or two kettles. One kettle will melt a large amount, however, if it is kept constantly full, since the molten asphalt in the kettle will quickly melt large blocks of the cold asphalt.

The accompanying section, Fig. 19, will aid in the illustration of the methods described.

J. R. Worcester, M. Am. Soc. C. E. (by letter).—The subject embraced by the paper is so comprehensive that the author has, of necessity, hastened over many features of the problem, a more extended account of which might have been interesting to the members of the Society. In the hope of shedding a little light on some of these points, the writer wishes to discuss briefly the portion of the paper which refers particularly to the train shed. Mr. Worcester.

Cantilever Principle.—To make more clear the reasons for adopting the cantilever style of trussing, it should be stated that before the method of supporting the roof was determined, the cross-section, so far as the roof line was concerned, was agreed upon almost exactly as it was finally built. This line was arranged so as to give a large vertical space for windows below the eaves and above the connecting roof, to enclose the whole width of the shed under a single roof and to avoid raising any portion so high as to overtop the head house. It was also determined that there should be two lines of intermediate columns placed substantially in the positions finally adopted.

With these conditions prescribed and with the manifest advantages of raising the bottom chord as much as possible, the cantilever principle naturally suggested itself as desirable. To ascertain whether it would result in any great economy, however, an estimate of the cost of trusses with the same outline, but broken over the interior columns, was made. It was found that the weight of trusses constructed on these lines would be about 10.7 lbs. per horizontal square foot, instead of 8.25 lbs., which was the weight of the cantilevers.

Whether the supported trusses could have been made as light as the cantilevers, had the shape been immaterial, was not determined; but it is doubtful, as the conditions, particularly the fact that only a small portion of the load was variable, were altogether favorable for cantilever construction.

Expansion.—The author has referred to the fact that only one expansion joint was provided in the trusses, and that in the central span; but in speaking of the intermediate columns as not anchored, he might be understood to mean that it was expected that motion might occur at the feet of these columns from changes of temperature. This was not the case, as it was recognized that the friction from the load on these intermediate columns would be so great as to render motion impossible, even if desirable. The side columns were made very stiff, and were calculated on the assumption that all the wind force would be transferred through them to the ground. Assuming, then, that the trusses were fixed at the outer end, it was expected that the intermediate columns must bend slightly as the length of the end spans varied with the temperature. Allowing for a motion of 1 in. per hundred feet, as the extreme effect of temperature, it was found that the strain caused by this motion in the intermediate column would not

Mr. Worcester. add above 25% to the compression caused by the total vertical load, and the combined strain would be only about 12 500 lbs. per square inch.

As a matter of fact, it seems that the allowance of 1 in. per hundred feet was excessive, for it was found that the total change at the expansion joint between a very hot summer day, when, before the covering was all applied, parts of the trusses were exposed to the direct sun, and an unusually cold winter day, before the shed was occupied and partly warmed as it will be by occupation, that is, under a range of 94° Fahr., the maximum contraction at the central expansion joint amounted to only 1⅛ ins. This is partly explained by the fact that the side posts, instead of being absolutely rigid, as assumed, allow a motion at the top of apparently about 1 in., making a total contraction of about 3⅛ ins., or $\frac{9}{16}$ in. per hundred feet.

In this connection, the writer can hardly agree with the author's conclusion with regard to the Midway Floor, that "the expansion of each piece of steel is apparently taken up in the riveted joints." It seems more probable that, as the extreme variation in temperature of this portion of the building probably does not exceed 40°, the elasticity of the material is called upon to compensate for any motion which might occur if the material were free to come and go.

General Design.—There are two points in the general design of the train shed which are not very clearly set forth in the paper or illustrations, and about which a word may not be out of place.

The trusses at the ends of the building, viz., *A*, *B*, *C*, *K*, and *L* are made the full depth of the end, with diagonals each extending over two of the panels indicated by the vertical lines on the cut.

These diagonals are omitted from Fig. 7, through an inadvertence, which the writer is informed will be remedied before the final publication. The verticals of these trusses act as beams to carry the wind pressure to the horizontal trusses which are situated in the plane of the top and bottom chords.

The reason for making the main monitor trusses with a 60-ft. span, disregarding the supports which might have been carried to the top of main trusses, was that these trusses are spaced more closely together than the main trusses, those intermediate between the main trusses being carried by the purlins on either side.

Pin and Rivet Connections.—Pin connections were adopted for the main trusses, and for much of the rod bracing, largely as a matter of convenience of erection and to improve the general appearance. There was, however, one very important detail where a riveted joint was used, namely, at the intersection of the bottom chord of the cantilever trusses with the intermediate columns. Here a pin would have been of such large diameter and the necessary restriction in metal of the post so great, to say nothing of the difficulty of having the chords

on the two sides of different widths, that a riveted joint was much more satisfactory, and it was adopted without hesitation. Mr. Worcester.

Length of Struts.—One feature of the design which was of considerable importance in the way of economy, but which was somewhat contrary to modern practice, was the ratio of length to diameter allowed in struts in riveted work. If rules such as that "the length shall not exceed 100 times the least radius of gyration" had been adopted, it is safe to say that the purlins and a large amount of the bracing members would have had to be made up of different shapes altogether from those used. The limit adopted was for the length not to exceed 50 times the least diameter. In the writer's opinion, the least diameter in such a limit is a more proper guide than the radius of gyration, as the stiffness of the member, which is the element most to be considered, more closely corresponds with the former. Whether the ratio of 50 is too great is open to debate, but where the members are straightened carefully after riveting, and again looked after when erected, and where, as in a roof, they are not liable to transverse forces of any kind, it seems a wise economy to use a liberal ratio.

Jack Rafters.—It was considered very desirable, in covering the roof, to arrange if possible a truly curved surface which should not show breaks over the purlins, as there are many points of view outside the building from which the line of sight would quickly detect any unevenness. It was therefore with some trepidation that the writer specified for the jack rafters 8-in I-beams, the span being about 20 ft. This was finally done, but the beams were given a camber greater than that required by the curve of the roof, by an amount (1 in.) which was approximately the theoretical deflection which the beam would have from the dead weight of the covering. The result of this was very satisfactory, the lines of the roof taking a very even curve.

In this connection, it is interesting to note that at the bottom of the slope the pitch is not less than 3 ins. per foot, the steepest which the writer has known of for a similar composition roofing. So far no evil effect has been observed.

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EXPERIMENTS ON THE PROTECTION OF STEEL AND ALUMINUM EXPOSED TO WATER.

Discussion.*

By Messrs. L. L. Buck, George Hill, F. W. Skinner, Thomas D. Pitts and George Tatnall.

Mr. Buck. L. L. Buck, M. Am. Soc. C. E.—Have any of the specimens been tested by being placed in salt water so as to be exposed part of the time, and part of the time not exposed ?

Some tests of paints on wires were made for the speaker by E. G. Freeman, M. Am. Soc. C. E. The wires were about No. 8 gauge and cut in lengths of about 6 ins. They were coated with different kinds of paint and were then fastened together snugly in bundles of seven. The interstices were filled and the bundles coated with paint outside. These bundles were fastened to a pile, under the resident engineer's office, midway between high and low water. They were protected from abrasion by floating matter, and were left in position for six months, but not in freezing weather.

The reason for making the test in this way was that the conditions obtained would approach, more nearly than any others, the conditions which would be met by the cables of the New East River Bridge. The wires were found to be corroded even inside, where the interstices had

* This discussion (of the paper by A. H. Sabin, Assoc. M. Am. Soc. C. E., printed in *Proceedings* for October, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

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been filled with the paint. There were only one or two substances which stood the test. One, called "cable sheet," the composition of which the speaker knows, but is not allowed to divulge, withstood abrasion and corrosion perfectly. This material was selected as the covering of the cables, although its application was somewhat difficult. Mr. Buck.

Has the author used Lucol oil in any of his experiments?

The speaker has had some trouble in having paint applied properly. Having secured an honest manufacturer and obtained from him the required paint, he has found that a further requisite was the right kind of a man to apply it. The speaker has had structures painted and has found that after six months the pigment could be rubbed off with the hand quite readily. In the next specification prepared by the speaker he has concluded to insert a clause requiring the paint manufacturer to have an inspector present all the time to see that the paint is properly put on. In this way it may be possible to fix the responsibility if the work is not satisfactory.

GEORGE HILL, M. Am. Soc. C. E.—The author has said that he considers these tests a fair criterion by which to judge of the sufficiency of the coating, as applied to structural material exposed to the air or the elements. What time relation is considered by him as existing between the exposure to fresh or salt water and the exposure which would exist, for instance, in the steel skeleton of a building surrounded with masonry? That is to say, if after an exposure of two years in water and one or two years to air most of the coating has disappeared, how many years will it take to produce a like result if the material is partially protected from corrosion? Mr. Hill.

Further, would all the protective coverings show the same relative deterioration under the conditions existing in buildings as they show when subjected to the direct action of fresh and salt water?

During the past six years the speaker has had Lucol oil under observation, and has used it quite extensively (for the last three years exclusively for exterior work and metal covering), and has found that it gave better results than any of the other oils. The speaker would like to ascertain the author's general views in regard to oil as compared with varnish. If the author's statements in regard to recent structural work in New York City are taken without qualification, it may be said that the life of the largest, or best, office buildings will be about twenty years, if the conditions under which the exterior supporting columns are installed are such as to subject these columns to a certain exposure; that is to say, if they are placed with their outer edges from 8 to 24 ins. from the exterior walls. The general practice is not to make any special provision for protecting the columns.

The speaker recalls one case in which it was specified that the columns should be left with a space around them. This space between

Mr. Hill. the column and the enclosing brickwork was to be filled in subsequently with Portland cement, which would be, without doubt, a sufficient protection if the column were not otherwise protected; but with a film of paint between the metal and the cement the speaker doubts the sufficiency.

In New York City the practice, which is nearly universal, is to use one of the paints or, occasionally, Smith's durable metal coating, but if these have a life of approximately only twenty years, the New York practice should change quickly.

The paper does not indicate what should be done, yet some lesson should be derived from these tests to show what the practice ought to be, for engineers do not want to build for only ten, fifteen or even twenty-five years.

Mr. Skinner. F. W. SKINNER, M. Am. Soc. C. E.—In New York City the general practice is to protect from moisture quite effectively certain parts of the steel work of office buildings. Most of the grillage foundations, for instance, are bedded in concrete, and frequently the metal surfaces are also plastered. The wall columns, in many cases, are only protected by paint, but in other cases there is a special protection.

In the St. Paul Building, for instance, there is a tile casing around the columns and also a coating of asphalt, or some kind of fibrous material wrapped around them.

Mr. Pitts. THOMAS D. PITTS, Jun. Am. Soc. C. E.—The author states that in these experiments the plates exposed in fresh water were placed vertically, while those exposed in salt water were placed horizontally. Was there any special reason for so placing the latter? There would necessarily be a deposit of silt on the upper surfaces of these plates which would measurably protect the coating. It would also prevent the growth of oysters, barnacles, seaweed, etc., which will not grow on surfaces where they cannot get a firm hold. Moreover, the silt deposit would not necessarily be uniform on all the plates, so that they would not all have the same degree of protection.

This being true, would not more uniform conditions be secured by placing all the plates vertically?

Mr. Tatnall. GEORGE TATNALL, M. Am. Soc. C. E. (by letter).—The subject of protective coatings for iron is one of great depth, the surface of which has been little more than scratched. The universal use of iron, for such diverse purposes divides the subject into a number of problems according to the exposure to which the iron is subjected.

The protection of ironwork exposed to ordinary out-of-door conditions of sunshine, rain, dew and variations of temperature is the most usual problem. The protection of ironwork in the indoor exposure incident to the roofs of trainsheds, foundries, shops and other manufacturing establishments, presents another and very different problem, wherein the absence of the direct heat and light of the sun, and the

drenching of rain, is offset by the presence of deleterious and corrosive gases. The protection of the skeletons of steel-framed buildings forms a third distinct problem. Mr. Tatnall.

Two other very similar problems, and very dissimilar to the others, are the protection of ironwork submerged in water, and ironwork alternately submerged and exposed, as by the fluctuation of tides. These problems present such dissimilar features that the results of tests under one, can by no means be taken as more than an indication of possible results under one of the others.

This paper presents some valuable and instructive data relative to the protection of iron completely submerged.

Charles B. Dudley, M. Am. Soc. C. E., Professor Spennrath and others have shown the effect of submersion in water on a linseed oil film, manifested in the softening, wrinkling and loss of adhesiveness of the film. This receives abundant confirmation in the blistering and separation of the coats, so frequently noted.

By far the greatest enemy to paint coatings on iron is the well-known and uncontrollable propensity of the metal to rust when in the presence of oxygen and moisture, which is so fierce as to produce, by some sort of endosmotic and exosmotic action, the rusting of the iron and deposition of the oxide on the outside, through the pores of protective coatings of other metals, such as tin, zinc or lead. Bright iron will not rust in an atmosphere of pure oxygen, nor will it rust when immersed in distilled water, and it is questionable whether the scanty amount of air ordinarily held in suspension by river or sea water would be sufficient to make the test as severe as it would be under conditions part wet and part dry, or even under those of ordinary outdoor exposure. In support of this, it can be stated that similar plates, to the knowledge of the writer, covered with two coats of some of the same paints mentioned in these tests, passed to complete destruction in from 6 to 12 months in ordinary outdoor exposure.

Almost invariably, the protective value, to iron, of a good paint, is destroyed in outdoor exposure by the penetration of moisture and air through the pores of the coatings; causing the formation of rust spots, microscopical at first, but increasing in size and number, and spreading and joining together under the film, until it is thrown off in flakes or shreds, long before its good qualities as a paint have disappeared. On this account, the very able prize essay of Professor Spennrath, complete and exhaustive as it is, in relation to the paint film, is useless as regards the protective value of that film to the iron beneath it.

The large number of samples, in these tests of submerged exposure, in which it was noted that the coatings had been destroyed and very little or no rusting manifested, would seem to indicate that the processes of destruction were different in this than in other exposures. The film seems to have been destroyed before the rusting commenced,

Mr. Tatnall. but how? The almost invariable presence of blisters, or separation of the two films from each other and from the metal, shows conclusively the same softening, loosening, and wrinkling action on these films, as occurred in the case of linseed oil films on glass submerged in water, in the experiments of Dr. Dudley. But it has certainly not extended so far as to effect complete separation of the films from each other or from the metal, or some half-peeled remnants would have been found.

The absence of any explanation for this rustless destruction of the film in submerged tests, as well as certain unexplained phenomena occurring in ordinary outdoor exposure, suggests the query, does linseed oil, by complete oxidation, by long submersion, by the presence of the naturally occurring chemical reagents, or by the combination of any or all of these causes, become to any extent soluble in water?

Although the varnish gums are exceedingly unreliable and uncertain, whether used on wood or iron, in ordinary exposures, it seems to be shown clearly by these tests that the addition of the best of them to linseed oil is of great benefit to the latter in resisting the soaking destruction of submersion, by whatever influences this may be caused.

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THE IMPROVEMENT OF A PORTION OF THE JORDAN LEVEL OF THE ERIE CANAL.

Discussion.*

By ALLEN HAZEN and GEORGE W. RAFTER.

ALLEN HAZEN, Assoc. M. Am. Soc. C. E.—There are, perhaps, few materials about which more different opinions are held than quicksand. A good definition of this substance is greatly to be desired. This paper contains a definition of quicksand which differs considerably from the idea which the speaker has entertained, and he will present briefly the idea which he has held, with the hope of starting a discussion leading to something more definite upon this subject. Mr. Hazen.

Mr. Landreth's definition of quicksand is: "A mixture of rounded particles of sand and clay, the sand predominating."

The speaker's idea of quicksand is: an even-grained sand, containing for the time more water than would normally be contained in its voids, and, therefore, with its grains held a little distance apart, so that they flow upon each other readily. The sand may be either coarse or fine, generally it is extremely fine. It is the speaker's idea that quicksand

* This discussion (of the paper by William B. Landreth, M. Am. Soc. C. E., printed in the *Proceedings* for December, 1899) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)
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Mr. Hazen. in general contains no clay. It may be that some materials contain a little clay, and still act as quicksand; but, if so, that they act as quicksand notwithstanding the clay, and not because of it. A material containing clay particles in considerable quantity is cohesive and impervious. Water may press it out of shape, make cracks in it and rush through it. Under some conditions the whole mass, under heavy pressure, may flow slowly like molasses, but with water it will never make an intimate mixture capable of flowing through small openings and behaving much like water, which is the characteristic property of quicksand.

The sand in a mechanical filter is a good illustration of quicksand. The sand is placed in a tub, with screens or other drainage apparatus at the bottom. The water flows downward through the sand during filtration. Occasionally, the flow is reversed to wash the sand. When the current is downward the sand is firm, and it remains firm after it is drained. If one steps upon it the track hardly shows. When the sand is washed by an upward current, it is lifted by the water, and occupies, perhaps, 10% more volume than it did with the downward current, and in this condition it is suspended in the water, and is so soft that a stick can be pushed into it with but little more resistance than would be offered by so much water.

As the voids in the sand are increased, the friction is greatly reduced, until a point is reached where the friction just balances the excess of weight of the sand over water, and this condition may be maintained indefinitely, the upward current of water just sufficing to hold the sand in a state of suspension.

In this condition it is ideal quicksand. The phenomenon is precisely the same whether the sand is of wind-worn spherical grains or of the most angular grains of crushed quartz. In either case the sand is made quick by the passage upward through it of a current of water so rapid that the friction which it encounters more than equals the weight of the sand, and as a result the sand is lifted. If the upward rate were somewhat less, the weight of the sand would exceed the friction, and the sand would not become quick, but would remain solid and firm, as with the downward current.

The upward velocity required to lift a sand in this way is a direct function of the size of the sand grains, and can be computed. The sand used in mechanical filters has an effective size of from 0.40 to 0.60 mm., and the velocity used in washing is such that the friction is more than equal to the excess in weight of the sand over water, but is not three times as great. If it were, some of the sand would be carried away.

Table No. 3 shows the computed velocities at which the friction equals the excess in weight of sands of various grain sizes, or, in other words, the velocities at which the sands will just be lifted.

Mr. Hazen.

TABLE No. 3.—COMPUTED VELOCITIES REQUIRED TO LIFT SANDS OF VARIOUS GRAIN SIZES.

At a temperature of 50° Fahr.

Effective size of sand.	Velocity of solid column of water, in meters per 24 hours.	Velocity of solid column of water, in inches per hour.
0.50 mm.	250	410
0.40 "	160	262
0.30 "	90	148
0.20 "	40	65
0.10 "	10	16
0.05 "	2.5	4
0.03 "	0.9	1.5

This table, perhaps, gives an indication of the reason why quicksands are usually fine sands. The finest mortar sand has an effective size of from 0.20 to 0.30 mm. To lift it, requires an upward velocity of from 5 to 12 ft. per hour, a velocity greater than those which generally occur in the ground water about excavations. That is to say, sand of this coarseness will only act as quicksand where the ground-water currents are unusually strong. With sand 0.10 mm. in diameter, a velocity of only 16 ins. per hour is required to lift it—a velocity which is probably quite common—while the lower velocities of 4 and 1.5 ins. per hour, required to lift sands with effective sizes of 0.05 and 0.03 mm., respectively, are almost sure to exist where excavations are made below the ground-water level in pervious materials; and where sands of these sizes exist they are almost sure to act as quicksands.

There is a condition which may make a sand quick which at first sight would seem to be different from that mentioned above, but which in reality is but a variation of it. It is when sand in apparent equilibrium is made quick by a sudden shock or blow. Let us suppose a layer of sand with an effective size of 0.05 mm. and 3 ft. deep, in which the voids are 42%, entirely filled with water. The grains are in a not very stable equilibrium, and this sand is capable of being compacted to 40% of voids. A smart blow or sudden pressure will disturb the equilibrium, and the sand will be suspended by the water which it contains. It will shrink 5% in going from 42 to 40% of voids; and, under the conditions assumed, if perfectly drained at the bottom, half an hour will be required for the excess of water to drain out of it. During this time it will be quicksand. This phenomenon may be seen on many lake shores where the sand is held full of water by capillarity, but in this case the sand is usually coarser, and the length of time that it remains quick is but a small fraction of the above, perhaps only a minute or two, or even less.

A number of samples of material, presumably quicksand, obtained by borings in connection with some of the deep waterways investiga-

Mr. Hazen. tions, and which were labeled as mixtures of clay and sand, have been handed to the speaker by Mr. E. P. North. Under the microscope these materials proved to be entirely free from clay, and consist of particles from 0.03 to 0.10 mm. in diameter, having effective sizes of approximately 0.04 mm. Ninety per cent. or more will pass a sieve with 200 meshes per lineal inch. These materials contain a little lime, but, so far as this is the case, the speaker is inclined to think that the lime tends rather to keep them from acting as quicksand than otherwise.

The question may be raised as to the propriety of extending the name of sand to these extremely fine materials. Materials of these sizes occur quite freely in Nature, in which the particles are mostly silica, occasionally with a mixture of hard silicates. Under the microscope they appear precisely like sand. The particles are angular, the arrangement of the particles and the percentage of voids are substantially the same as with coarse sands. The relation of these materials to ordinary sand is much the same as that of sand to gravel, but it is not correct to speak of sand as fine gravel, and it may not be correct to speak of these materials as fine sand. The terms silica dust and sand dust have been suggested, but they imply dryness, and do not seem suited to quicksand. The word silt is also used, but this suggests a somewhat different meaning. Microscopic sand would perhaps be a better term.

Clay is an entirely different substance. Mr. H. W. Wiley,* Chemist of the United States Department of Agriculture, makes the following statement in regard to the properties of clay:

"The percentage of pure clay is about 75% in natural clays, 45% in heavy clay soils, and 15% in ordinary loamy soils. When freshly precipitated by brine it is gelatinous, resembling a mixed precipitate of iron and aluminum oxides. Its volume greatly contracts on drying, clinging tenaciously to the filter, from which it may be freed by moistening. On drying it becomes hard, infriable and often resonant. It usually possesses a dark brown tint, due to iron oxide. Under the action of water it swells up like glue, the more slowly as the percentage of iron is greater. In the dry state it adheres to the tongue with great tenacity. According to Whitney the finest particles of colloidal clay have a diameter of 0.0001 mm. With a magnifying power of 350 diameters, however, Hillgard states that no particles can be discerned."

The clay particles are tens, if not hundreds, of times smaller than the smallest sand grains here considered, and differ from them, both physically and chemically.

It is the speaker's impression that there is a good deal of looseness in distinguishing between clay and microscopic sand. Sand is often so fine as not to be gritty, and when moist it has many of the properties of clay. It differs from clay in its lack of adhesion when dry. A very small percentage of clay, however, makes it adhesive.

* "The Principles and Practice of Agricultural Analysis," p. 282.

The speaker thinks that in many instances microscopic sand, either entirely or nearly free from clay, has been mistaken for clay. So far as he knows, nearly all clay contains more or less microscopic sand, and the percentage of sand may become quite large before it ceases to be called clay. The microscope at once reveals the difference between clay and sand, and there is no good reason for confounding them. Mr. Hazen.

GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—In discussing this paper the writer recognizes that Mr. Landreth was not in any degree responsible for the plans adopted, but that he is historian merely of what, for lack of thorough knowledge of the conditions, turned out to be an exceedingly unsatisfactory piece of construction. Mr. Rafter.

As to why this particular construction was so unsatisfactory, the writer will not now attempt to determine. The discussion of that question pertains rather to a broad history of the Erie Canal, in which the results of many years of management of a great public work on political lines are traced to final philosophical conclusions. This part of the subject is of extreme interest and could be expanded indefinitely. Nevertheless, the writer leaves it untouched any further than to remark that the absence of systematic boring records along the Erie Canal probably led to some serious errors of omission.

The methods finally adopted are detailed clearly in the paper. Taken in conjunction with the long struggle against the inevitable, which preceded their adoption, they have seemed to the writer to indicate that, from first to last, this work was conducted on experimental lines purely. Apparently, no one quite grasped the real scope of the problem presented. In order to indicate the basis for this position, let us outline the physical conditions to be met.

As indicated in the paper, the marl varies in depth from 2 to 15 ft. Beneath this is found soft clay to a depth of 40 to 50 ft. from surface of ground. The surface soil is swamp muck.

Such conditions indicate clearly that margins of excavations should be kept clear of extraneous loads. Nevertheless, as shown by the photographs, this precaution was ignored. Even after a year's experience the contractors were allowed to weigh down the margins of drainage ditches with freshly excavated material. The sliding of the banks of these ditches is therefore merely an illustration that like causes produce like results.

It is clear to the writer, therefore, that the first thing to be done was to clear the margins of excavated material. The next step was to remove the muck above the marl for some distance back on either side of the main channel. After this was done the deepening of the channel, even for several feet, would have been a very simple matter. The slopes would properly have been made flat, in this system of construction.

If embankments over such material are necessary, the proper pro-

Mr. Rafter. cedure is to strip the marl for 50 to 100 ft. on each side of the channel, and construct the embankment with a berm 10 to 20 ft. wide on the inside. In this way the writer believes that a canal can generally be constructed through marl without special extra expense, other than for wide right of way. In the present case, if it is deemed necessary to maintain towing paths on the original lines, a timber platform on piles will answer every purpose.

From near the foot of Cayuga Lake to some distance below Mosquito Point, Seneca River flows over marl beds, and from the New York Central and Hudson River Railway Viaduct to Mosquito Point, a new channel was cut in this material about 25 years ago. This channel extends from 6 to 8 ft. into marl, and its banks stand at a slope of about 1½ to 1. In 1858, or thereabout, a new channel for Canandaigua Outlet was also cut through Seneca River marl in the vicinity of Montezuma Aqueduct, which has not given any trouble by the rising of the bottom, such as perplexed the Erie Canal engineers at Warners, in 1896–97. The writer cannot but think, therefore, that a study of the extensive work actually carried out in marl in the vicinity of Jordan Level, would have indicated the proper methods of construction to use in that material.

In regard to the expensive method of piling and cross-bracing finally adopted, the writer understands that it has been only moderately effective. Slides of the slopes still occur. As to the extent of these, it is hoped that Mr. Landreth will give an account in his final discussion.

Vol. XXVI. MARCH, 1900. No. 3.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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SOCIETY AFFAIRS.

PAPERS AND DISCUSSIONS.

ILLUSTRATIONS.

The prices of publications are as follows: *Proceedings*, $6 per annum; *Transactions*, $10 per annum. Postage will be added when *Proceedings* are sent to foreign countries.

American Society of Civil Engineers.

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ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 A.M. to 10 P.M.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - - - 588 Columbus.
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Vol. XXVI. MARCH, 1900. No 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

March 7th, 1900.—The meeting was called to order at 8.40 P. M., Vice-President Rudolph Hering in the chair; Charles Warren Hunt, Secretary, and present, also, 97 members and 10 visitors.

A paper by Charles D. Marx, M. Am. Soc. C. E., Charles B. Wing, Assoc. M. Am. Soc. C. E., and Leander M. Hoskins, C. E., entitled "Experiments on the Flow of Water in the Six-Foot Steel and Wood Pipe Line of the Pioneer Electric Power Company at Ogden, Utah, Second Series," was presented in abstract by Emil Kuichling, M. Am. Soc. C. E., and was discussed by Messrs. E Kuichling and G. C. Whipple.

Ballots were canvassed and the following candidates declared elected:

As Members.

Alonzo Clarence Bell, New Orleans, La.
Edward Arnold Correa, Parral, Mex.
Philip Embury Harroun, Albuquerque, N. Mex.
Charles Keller, St. Louis, Mo.
John Millis, Willets Point, N. Y.
William Parker, Boston, Mass.
Herbert Ellwood Sherman, Providence, R. I.

As Associate Members.

Harry Thomas Cory, Columbia, Mo.
Robert Morris Drake, San Francisco, Cal.
George Ezra Ellis, Albany, N. Y.
Peter Platter Evans, Boston, Mass.
Theodore William Hill, Charleston, W. Va.
Henry Basil Magor, New York City.
Walter Sothoron Moore, Cleveland, Ohio.
Juan Tonkin Thomas, Iquique, Chile.
Irving Sparrow Wood, Providence, R. I.
Robert Patterson Woods, Wabash, Ind.

Announcement was made that the following candidates were elected by the Board of Direction, March 6th, 1900:

As Associate.

Wolfgang Gustav Triest, New York City.

As Juniors.

Guy Whitmore Culgin, New York City.
George Wallace Enos, New York City.
Charles Edward Howe, Wabash, Ind.
John Burton Stouder, West Albany, N. Y.
Sutton Van Pelt, Sault Ste. Marie, Mich.
Levi Romulus Whitted, Norfolk, Va.

The Secretary announced that at the meeting of the Board of Direction, March 6th, 1900, the ballot on the reconsideration of Allen Hazen was canvassed, and that Mr. Hazen was declared elected as a Member of the Society.

The Secretary announced the death of ERNEST GREY FREEMAN, elected Associate Member May 3d, 1893; Member October 5th, 1898; died March 6th, 1900.

Adjourned.

March 21st, 1900.—The meeting was called to order at 8.40 P. M., Vice-President Rudolph Hering in the chair; Charles Warren Hunt, Secretary, and present, also, 93 members and 17 visitors.

A paper by George S. Webster and Samuel Tobias Wagner, Members Am. Soc. C. E., entitled "History of the Pennsylvania Avenue Subway, Philadelphia, and Sewer Construction Connected Therewith," was presented by Mr. Wagner, and illustrated with lantern slides.

The subject was discussed by Messrs. Charles Macdonald, A. C. Gildersleeve, A. P. Boller, L. L. Buck, O. Lowinson, Rudolph Hering and the authors.

The Secretary announced the death of EDWARD BATES DORSEY, elected Member June 4th, 1879; date of death not known; and of EDWARD HIGGINSON WILLIAMS, elected Member September 5th, 1883; died December 21st, 1899.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

March 6th, 1900.—The Board met at 8.45 P. M., Vice-President Hering in the Chair; Charles Warren Hunt, Secretary, and present also Messrs. Bensel, Buchholz, Deyo, Knap, O'Rourke, Ricketts, Seaman, Turner and Whinery.

A report was received from the Library Committee, and the Committee was authorized to prepare and publish a Catalogue of the Library.

Ballots were canvassed in the matter of the reconsideration of the ballot on the application of Allen Hazen for admission as Member, and Mr. Hazen was declared elected.

Applications were considered, and other routine business transacted.

Adjourned.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 A. M. to 10 P. M.

MEETINGS.

Wednesday, April 4th, 1900, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by William B. Landreth, M. Am. Soc. C. E., entitled, "Recent Stadia Topographic Surveys: Notes Relating to Methods and Cost," will be presented for discussion. This paper is printed in the current number of *Proceedings.*

Wednesday, April 18th, 1900, at 8.30 P. M., a regular meeting will be held, at which a paper by George W. Rafter, M. Am. Soc. C. E., entitled, "On the Flow of Water over Dams," will be presented for discussion. This paper is printed in the current number of *Proceedings.*

Wednesday, May 2d, 1900, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by J. M. Moncrieff, M. Am. Soc. C. E., entitled, "The Practical Column under Central and Eccentric Loads," will be presented for discussion. This paper is printed in the current number of *Proceedings.*

ANNUAL CONVENTION.

The returns so far show that 59 persons connected with the Society, accompanied by 43 guests, a total of 102, will sail from this country in time to be present at the Convention. Many of our foreign members also, have stated that they will attend. The Publication Committee now has under consideration, and will soon announce, several subjects for discussion at the Convention. In choosing these subjects an effort will be made to select such as will be of interest to engineers of both countries, for the purpose of securing an interchange of views.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(February 12th to March 14th, 1900.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c
(14) *The Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor. 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*. Stockholm, Sweden.
(57) *Teknisk Ugeblad*, Christiania, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

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Operating Mechanism, Duluth and Superior Swing Bridge. (14) Feb. 22.
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The New Colenso and Frere Bridges. (12) Feb. 16.
The Interprovincial Bridge, Ottawa, Canada. (14) March 10.
The Cuyahoga River Rolling Lift Bridge. (14) March 3.
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Ponts Roulants Électriques. (34) Serial beginning Jan., ending Feb., 1900.
Le Pont-Canal de Briare. (35) Serial beginning Jan., ending Feb., 1900.

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The Standardization of Electrical Engineering Plant. R. Percy Sheldon. (12) Feb. 9; (28) Feb. 9.
The Facilities Afforded by the U. S. Office of Standard Weights and Measures for the Verification of Electrical Standards and Electrical Measuring Apparatus. Frank A. Wolf, Jr. (27) March 10.
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City Road Electric Supply Station. (12) Feb. 23.
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Economic Equipment of Electric Stations. Alton D. Adams. (27) March 3.
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Shipbuilding and Marine Engineering in 1899. Benjamin Taylor. (9) March, 1900.
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The Oliverson-Killingbeck Variable Speed Gear. (28) Feb. 16.
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Construction of the St. Louis, Peoria & Northern Railway. F. G. Jonah. (13) Feb. 15.
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Report on Sewage Sludge Deposits on the Experimental Coke Beds, London, England. (13) March 8.
Les Puisards à Chicanes et les Puisards de Rue. A. Stas. (30) Feb., 1900.

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Some Practical Notes on the Manufacture of Steel on the Basic Open Hearth. Thomas Turner. (22) Feb. 23.

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The Present Status of Engineering Knowledge Respecting Masonry Construction David Molitor, M. Am. Soc. C. E. (1) Jan., 1900.
Tests of Small Model Cement Arches to Determine the Value of Tie Rods. Daniel B. Luten. (13) Feb. 15.
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The Designing of Skew Connections for Roof and Bridge Work, C. A. P. Turner. (13) Serial beginning Feb. 15, ending Feb. 22.
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Water Supply.

An Electric Recording River Gauge. (14) March 10.
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A British View of Water Waste. (14) Feb. 17.
Water Meters of the Present Day with Special Reference to Small Flows and Waste in Dribbles. William Schönheyder. (11) Serial beginning Feb. 2, ending Feb. 9; (47) beginning Feb. 3, ending Feb. 10.
The Efficiency of Mechanical Filtration. (14) March 10.
The Water-Works of Merthyr Tydfil. (14) March 10.
Essai de Terminologie Hydrologique. Th. Verstraeten. (31) Fourth part, 1899.
Quelques Considerations sur la Production de l'Ozone et son application à la Stérilisation des Eaux. M. X. Gosselin. (32) Feb., 1900, No. 3.

Waterways.

The Silting-up of Lake McDonald and the Leak at the Austin Dam. Thomas U. Taylor, Assoc. M. Am. Soc. C. E. (13) Feb. 22.
A 6-Cubic Yard Dipper Dredge for Use on the Great Lakes. (13) March 1.
Proposed Improvements of the Southwest Pass at the Mouth of the Mississippi. (13) Feb. 15.
New York's Canal Problem—A Discussion of the Report of the Advisory Committee. William G. Raymond, M. Am. Soc. C. E. (15) Serial beginning March 2, ending March 9.
Dock Equipment for the Rapid Handling of Coal and Ore on the Great American Lakes. Arthur C. Johnston. (1) Jan., 1900.
Le Port d'Escale de Zeebrugge. Ch. Piens. (30) Feb., 1900.
Le Port de Bilbao. Alfonso Dory. (31) Serial beginning in fourth part, 1898, ending in fourth part, 1899.

MEMBERSHIP.

ADDITIONS.

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		Date of Membership.
BEARD, EDWARD JAMES, Prin. Asst. Eng., Choctaw, Oklahoma and Gulf R. R. Co., Little Rock, Ark		Feb. 7, 1900
HAZEN, ALLEN, Cons. Eng., St. Paul Bldg., New York City	Assoc. M.	June 3, 1896
	M.	Mar. 6, 1900
MILLIS, JOHN, Willets Point, Borough of Queens, N. Y		Mar. 7, 1900
OLMSTED, FRANK HENRY, City Engineer, Los Angeles, Cal		Feb. 7, 1900
PARKER, WILLIAM, Division Eng., Boston & Albany R. R., Room 372, South Station, Boston, Mass		Mar. 7, 1900
QUINTON, JOHN HENRY, Consulting Eng., City Engineering Dept. (Res. 1016 West 8th St.), Los Angeles, Cal		Feb. 7, 1900

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	Assoc. M.	Feb. 7, 1900
EVANS, JOHN MAURICE, Inspector, N. Y. C. & H. R. R. R., Albany, N. Y	Jun.	May 4, 1897
	Assoc. M.	Jan. 3, 1900
EVANS, MYRON EDWARD, 20 Nassau St., New York City	Jun.	Jan. 3, 1895
	Assoc. M.	Feb. 7, 1900
FISH, JOHN CHARLES LOUNSBURY, Assoc. Prof. of Civil Eng., Stanford Univ., Palo Alto, Cal	Jun.	Jan. 31, 1893
	Assoc. M.	Feb. 7, 1900
HARRINGTON, JOHN LYLE, 35 Crescent St., Middletown, Conn	Jun.	Aug. 31, 1897
	Assoc. M.	Oct. 4, 1899
JOHNSON, RANKIN, Chf. Asst. Eng., Mexican International R. R., Durango, Mexico		Feb. 7, 1900
MOLER, WILLIAM GRIFFITH, Apartado 289, Mexico City, Mexico		Jan. 3, 1900
SCHMITZ, FRANK CURTISS, 908 Lawyers' Bldg., Newark, N. J		Dec. 6, 1899
WOOD, IRVING SPARROW, Asst. Eng., City Engrs. Office, Providence, R. I	Jun.	Mar. 5, 1890
	Assoc. M.	Mar. 7, 1900

ASSOCIATE.

	Date of Membership.
WEBER, GEORGE ADAM, Empire Bldg., 71 Broadway, New York City	Feb. 6, 1900

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HOWE, CHARLES EDWARD, Chf. Eng., Wabash Bridge and Iron Works, Wabash, Ind.	Mar. 6, 1900
WADSWORTH, GEORGE REED, Asst. Eng., Middle Div., N. Y. C. & H. R. R. R. Co, Albany, N. Y.	Feb. 6, 1900

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ROGGE, JOHN CHARLES LEWIS	683 East 141st St., New York City.
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WEISKOPF, SAMUEL C.	Battery Park Bldg., 24 State St., New York City.

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Hill, Walter Arthur.............1425 Spruce St., Berkeley, Cal.
Jordan, William Frederick . .....Care of Chief Engrs. Office, N. Y. C. & H. R. R. R., Grand Central Station, New York City.
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Lockwood, Willard Datus ....83 Meigs St., Rochester, N. Y.
McMeekin, Chas. Wm..............P. O. Box 821, Vancouver, B. C.
Mogensen, Olaf Einar..... ... ...Box 235, Winnetka, Ill.

JUNIORS

Fisher, Wager.....................Care of Jersey City Water Supply Co., Boonton, N. J.
Gregory, John Herbert ....Asst. Eng. Imp. of Water Supply, Bureau of Highways, 418 City Hall, Philadelphia, Pa.
Horton, Sandford. ....214 N. Summit St., Indianapolis, Ind.
Millard, Curtis................ ...Gen. Man., C. P. & St L. Ry. Co. of Illinois, Springfield, Ill.

DEATH.

Freeman, Ernest Grey ....Elected Associate Member May 3d, 1893; Member October 5th, 1898; died March 6th, 1900.
Williams, Edward Higginson......Elected Member September 5th, 1883; died December 21st, 1899.

Vol. XXVI. MARCH, 1900. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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Vol. XXVI. MARCH, 1900. No. 3.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

RECENT STADIA TOPOGRAPHIC SURVEYS: NOTES RELATING TO METHODS AND COST.

By WILLIAM B. LANDRETH, M. Am. Soc. C. E.

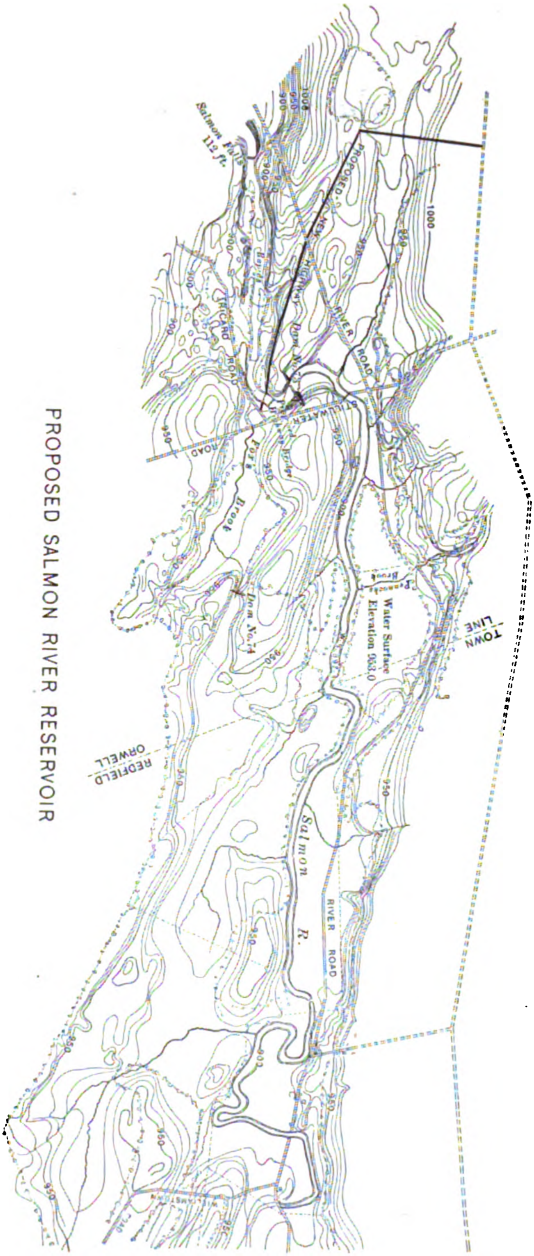
TO BE PRESENTED APRIL 4TH, 1900.

Surveys to determine the location of possible reservoirs and conduit lines for furnishing water for the summit level, or Rome level, of the proposed Deep Waterway on the Oswego-Mohawk-Hudson route were made under the direction of the Board of Engineers* on Deep Waterways, between August 1st, 1898, and June 1st, 1899. George W. Rafter was Chief Engineer of the Water Supply Divisions, with J. Y. McClintock, H. F. Northrup and the writer, Members Am. Soc. C. E., as Assistant Engineers.

Mr. McClintock was in charge of the surveys on the feeder line from Carthage to Rome, N. Y. Mr. Northrup was in charge of the base-line surveys on the Salmon and Black Rivers, and of the topographic work on the latter stream near Carthage. The writer was in charge of the topographic work on the Salmon River, on the Black River beyond the limits of Mr. Northrup's survey, and on a portion of the feeder line, as a branch of Mr. McClintock's main party. Mr. B. E. Failing was

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited, from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

*Lieut.-Col. C. W. Raymond, Alfred Noble and George Y. Wisner, Members, Am. Soc. C. E.



PROPOSED SALMON RIVER RESERVOIR

FIG. 1.

transitman, and Mr. J. T. Parsons in charge of the office and map work, on the writer's party.

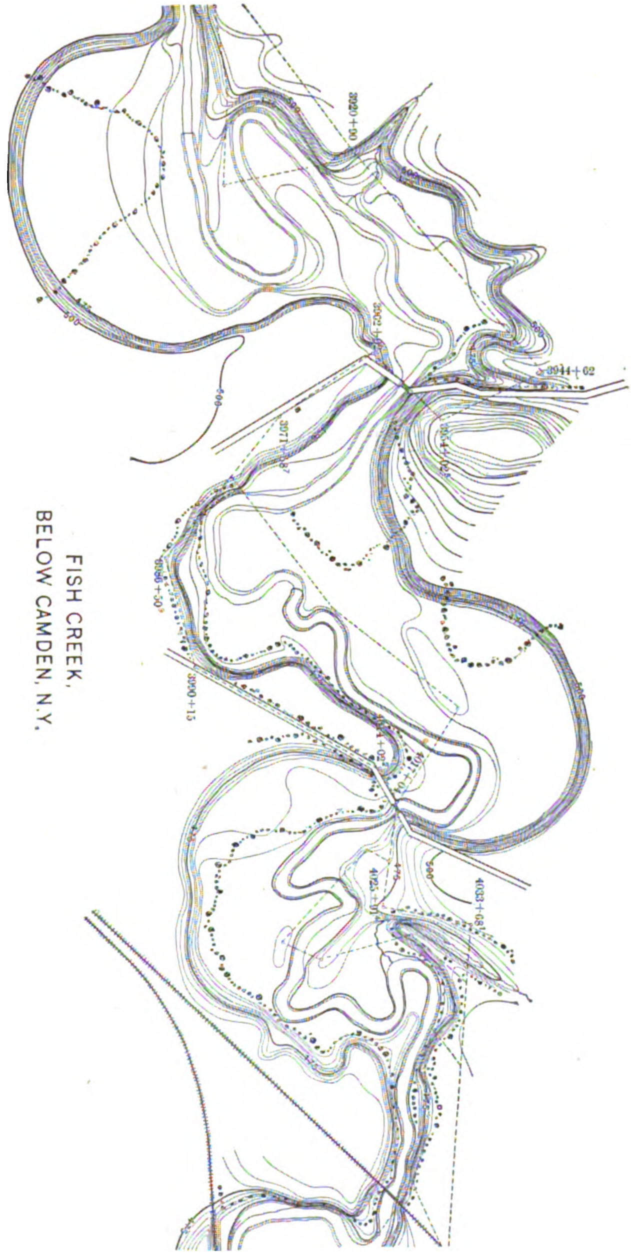
Party and Outfit.—The topographic party was composed of an assistant engineer in charge, a transitman, a recorder, three or more stadia rodmen, a draughtsman, a computer and two or more axmen. When running spirit levels, Mr. R. L. Burns, head stadia rodman, was detached from the main party for that purpose.

The outfit consisted of a transit fitted with non-adjustable stadia hairs, its horizontal limb reading to 20 seconds, and vertical arc to single minutes, one stadia rod for each rodman, a Y-level and rod; a large canvas umbrella, a tin megaphone, a 100-ft. steel tape, and the necessary office outfit of drawing table, instruments, etc. The whole outfit had been in use previously on the surveys of the main line of the Deep Waterway.

Field Work.—The method used in the topographic field work was that generally adopted on surveys where the distances are taken by stadia. The assistant engineer selected the base-line points to be occupied, and located the forward stadia points on the circuits, so as to cover the territory fully, and the party followed as closely as possible. Each rodman was given a particular class of objects to locate; one following the streams, another taking contour points, another the roads, buildings, woods, etc.

Each rodman worked independently, but when convenient all were kept on the same side of the transit, to facilitate the reading and plotting of the azimuths. An early trial was made of the method of placing all the rodmen on one azimuth, taking readings to each, and then moving all of them ahead to another azimuth; but it was soon abandoned as being productive of vexatious delays.

In working on streams of considerable size a circuit was run along one bank, and both slopes were worked from it, the party of rodmen being divided between them. Wooded gorges were worked from circuits on the top of each bluff, each slope being taken from the circuit on the opposite bluff. When the large streams were frozen, a circuit was run on the ice, and both banks were located therefrom. Crooked unfrozen streams were located from a circuit crossing them at the bends, with rodmen on each bank, the remainder of the party crossing from point to point in a boat. Long wooden slopes were taken from circuits along the top and foot thereof, and from short circuits con-



FISH CREEK,
BELOW CAMDEN, N.Y.

FIG. 2.

necting them, and so spaced as to cover the territory fully. Two corners of each building were generally located, the rodman measuring the other dimensions with his rod and giving the data to the recorder when passing him.

All streams, roads, buildings, outlines of woods, town and county lines and changes of slope were located. The contour interval on the surveys of the Salmon and Black River reservoirs was 10 ft., and on the Fish Creek feeder line 5 ft., with a 2-ft. interval on 2 square miles near the dam site on the Salmon River.

In rough ground the assistant engineer made field sketches on stiff-backed pads of cross-section paper, upon which governing points were located and numbered to correspond with the numbering of the shots locating them. It was found that the making of full sketches on all kinds of territory greatly retarded the progress of the field work without a corresponding gain in the rate of mapping.

Office Work.—At the close of each day's field work the field party reduced and checked the day's stadia readings and vertical-angle elevations, and the office force calculated the latitudes and departures of the stadia circuits. Two or more field books were in use at one time, the recorder changing books each day and leaving the book of the previous day for the use of the office force.

The mapping sheets were of heavy mounted white paper, 45 x 32 ins., with a working limit of 37 x 25 ins. The base, sub-base and stadia circuit lines were plotted carefully by latitudes and departures, care being taken to locate the working borders of each sheet so that the mapped surface would be evenly balanced thereon.

The sheets were lettered, inked and tinted to correspond with the same class of work done by the United States Geological Survey. The scale of the Salmon River and Black River maps was $\frac{1}{10000}$, and of the Fish Creek map $\frac{1}{5000}$.

Description of Territory.—The work on the Salmon River consisted of a survey of the valley of that stream above the Salmon River Falls, for a possible reservoir site. For a mile above the falls the gorge of the river is from 400 to 500 ft. wide with steep banks from 75 to 100 ft. high.

At the head of the gorge the valley widens out, forming flats from 1 to 1½ miles wide, which extend about 10 miles up stream.

The river flats are 900 to 910 ft. above tide water, the survey cover-

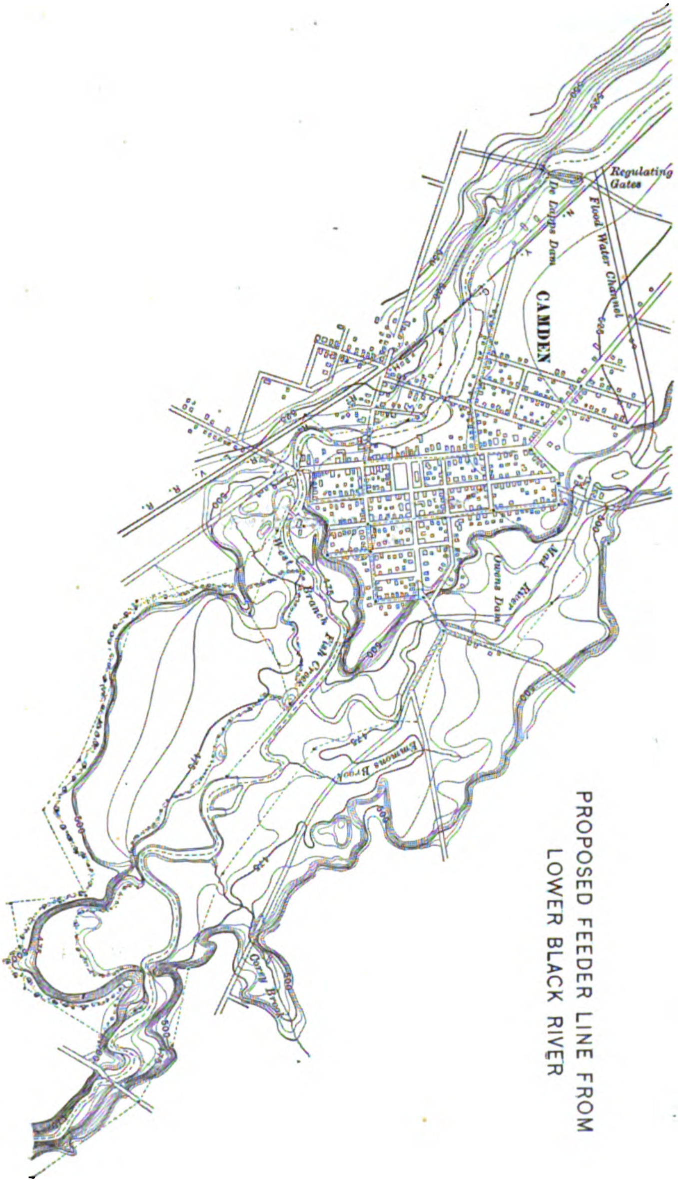

Regulating Gates
De Lapps Dam
Flood Water Channel
CAMDEN
West Branch
Fish Creek
Mad River
Emmons Brook
PROPOSED FEEDER LINE FROM
LOWER BLACK RIVER

FIG. 8.

ing the country up to the 970-ft. contour. About 65% of the area surveyed is covered with small second-growth timber and swamps. The north slope of the valley is an unbroken range of hills, but the south slope is broken, necessitating the careful location of, and spirit leveling over, several long dyke sites. A range of partly wooded hills, from 50 to 75 ft. high runs lengthwise of the valley on the south side of the river, for a distance of 5 miles, with dense swamps between it and the south side. The village of Redfield, having a population of about 500, came within the limits of the survey.

The survey of the Fish Creek Valley extended from 2 miles above the village of Williamstown to 2 miles below the railroad station at Taberg, a distance by rail of 21 miles. The survey covered the flat portion of the valley and the side slopes and tributary streams to an elevation of 75 ft. above the stream. The ground covered was mostly grazing and farm lands, 40% of which was timbered. Below the village of Camden the stream runs in a crooked gorge from 200 to 600 ft. wide, having steep wooded walls from 75 to 150 ft. high, cut from an open plain. The villages of Williamstown, West Camden, Camden and McConnellsville lie within the area covered by the survey.

The survey for a reservoir site on the Black River covered the valley of that stream between the villages of Carthage and Lyons Falls, up to the 790-ft. contour. At Carthage the sides of the valley approach to within 500 ft. of each other, forming a natural dam site, with rock at a reasonable depth. Above Carthage the valley widens out, until at Castorland, 10 miles up stream, it is 8 miles wide. Opposite the village of Lowville, 19 miles from Carthage, the valley begins to narrow up, and at Lyons Falls, 40 miles from Carthage, the sides close together, forming falls 65 ft. high. Three large streams enter the valley from the east, forming long tributary valleys.

The river flats lie mostly between the 730 and 740-ft. contours; the river being canalized between Carthage and Lyons Falls, with dams and locks at Otter Creek and Bushee's Landing.

About 25% of the area covered by the survey is wooded, and the east side is partly covered with granite boulders, ranging in size from that of a barrel to that of an average city block.

An area of 15 sq. miles above the 790-ft. contour was surveyed for a relocation of the Black River Division of the New York Central and Hudson River Railroad, between the villages of Carthage and Lowville.

Accuracy of the Surveys.—As the use to which the finished maps were to be put did not call for great accuracy in the surveys, it was not sought. The stadia circuits checked within 15 ft. in latitudes and departures, and within 0.50 ft. in elevation, with a permissible limit of 1 ft. in the latter. The error on closure in the stadia circuits was from 1 in 1 200 to 1 in 1 400, their length varying from ½ mile to 8 miles. A 30-mile circuit of spirit levels closed within 0.09 ft., being within the limit of error allowed on the United States Geological Survey.

Rate of Progress and Cost.—Table No. 1 gives the rate, area covered, cost per square mile, and other data regarding the entire work.

TABLE No. 1.—Rate, Cost and Other Data of the Three Surveys.

Survey.	Dates.	Number of days worked.	Set ups.	Shots.	Miles in stadia circuit.	Old base line occupied. Miles.	New base line. Miles.	Spirit levels, in miles.	Area, in square miles.	Square miles per day.	Cost.			
											Field work, per square mile.	Map work, per square mile.	Total, per square mile.	Total.
											$	$	$	$
Salmon River.	1898, Oct. 22d, Dec. 9th.	47	771	8 838	53.97	25.85	.....	1.92	15	0.32	66.00	14.00	80.00	1 200.00
Fish Creek.	1898, Dec. 13th, 1899, Feb. 3d.	44	451	11 776	56.18	24.87	.....	.....	19	0.45	54.00	25.00	79.00	1 500.00
Black River.	1899, Apr. 4th, May 27th.	47	600	11 166	188.48	34.40	17.80	56.66	85	1.81	16.50	7.00	23.00	2 000.00

The cost includes the salaries, maintenance, traveling expenses and supplies for the entire party, from the time of leaving the main office until the completion of the finished maps.

The cost of the base-line surveys for the Salmon River and Fish Creek work, and for one-third of the Black River, is not included in the cost as given. The area given in the table is that covered by the writer's party. The variation in the cost and rate per square mile on the several surveys is probably caused by the different conditions of weather, foliage and territory covered by them, and the increase for mapping on the Fish Creek survey is traceable to the use of a small contour interval and a large map scale.

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PAPERS AND DISCUSSIONS.

ON THE FLOW OF WATER OVER DAMS.

By George W. Rafter, M. Am. Soc. C. E.

To be Presented April 18th, 1900.

The classical experiments of the late James B. Francis, M. Am. Soc. C. E., on the flow of water over sharp-crested weirs, extended our knowledge of the general problem of weir flow considerably; and although Mr. Francis pointed out the fact that flow over sharp-crested weirs followed quite different laws from those of flow over broad and sloping crests, nevertheless it is probably true that 95% of all computations of flow over dams, made in the United States in the last twenty-five years—whatever the form of crest—have been based upon Mr. Francis' formula for sharp-crested weirs. So far has this erroneous practice proceeded that engineers have even used Mr. Francis' sharp-crested weir formula for computing flow over irregular profiles, because, in cases of litigation, Courts would accept the results without question. This is the more extraordinary because Mr. Francis himself showed, by his study of the Merrimac Dam, the considerable variation in flow resulting from change of form of crest.

Probably the main reason engineers have gone so far astray on this question has been the nearly entire lack of data applying to various

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in the volumes of *Transactions*.

forms of crests. It is true that a few advanced hydraulicians have discussed the effect of variations in width of crest, but it has remained for H. Bazin, *Inspecteur Général des Ponts et Chaussées*, to elucidate the whole problem in a series of masterly studies, which may be found in *Annales des Ponts et Chaussées* for the years 1888, 1890, 1891, 1894, 1896 and 1898. These studies are, as regards detail and minute research, unparalleled.

Messrs. Fteley and Stearns,* have indeed shown the effect of width of crest on discharge over weirs. Their experiments, while decisive for the cases studied, are still quite limited in scope, but Bazin has determined coefficients for a very large number of cases, not only of crests of different widths, but with varying front and rear slopes, as well as for curved profiles. Indeed, taking into account the backward state of knowledge of flow over weirs, his work is in many respects revolutionary. Certainly, with the data now available, there is no excuse for using a sharp-crested weir formula in computations of flow over various-shaped crests, irregular or otherwise.

In making the foregoing comments, the writer has no intention of criticising the work of others. Indeed, he has himself groped blindly in the dark in this matter, as other engineers have done. His intention is to present an irrational practice so saliently that—with the data now at hand—from this time on, the use of sharp-crested weir formulas for computing flow over weirs of all sorts of shapes will be discontinued.

An illustration from the writer's experience is pertinent to the discussion. During the last twenty years he has had occasion to gauge streams extensively in various parts of the United States. Long experience convinced him several years ago that sharp-crested weir formulas were not applicable to dams with sloping crests, and accordingly, in arranging for an extensive series of gaugings of the Genesee River, over a sloping-faced dam at Mount Morris, New York, in 1893, he used the formula

$$Q = 3.4\, L\, H^{\frac{3}{2}}\dagger$$

as originally proposed for this dam by Augustus S. Kibbe, Jun. Am. Soc. C. E., in his report on Genesee River Storage, to John Bogart, M. Am. Soc. C. E., formerly State Engineer and Surveyor.

In 1896, after the Genesee River gaugings had been in progress at

* "Experiments on the Flow of Water, made during the Construction of Works for Conveying the Water of Sudbury River to Boston." By A. Fteley and F. P. Stearns, *Transactions*, Am. Soc. C. E., Vol. xii, p. 1.

† See Appendix F to Annual Report, State Engineer and Surveyor, for 1890, p. 455.

the dam of the Mount Morris Hydraulic Power Company for three years, a sharp-crested weir was erected on the Genesee River about 2½ miles above, where rock bottom clear across the river afforded an opportunity for such construction without heavy expense.*

TABLE No. 1.

H = head on weir, in feet.	h = head on dam, in feet.	Computed discharge over weir, in cubic feet per second, for heads = H.	Computed discharge over dam, in cubic feet per second, for heads = h.	Percentage differences in discharges.
(1)	(2)	(3)	(4)	(5)
0.50	0.60	185	135	—37.0
0.70	0.83	310	330	+ 6.0
0.80	0.90	450	445	— 1.0
1.02	1.00	540	505	— 7.0
1.86	1.55	1 325	1 260	— 5.0
2.01	1.75	1 490	1 605	+10.0
2.42	2.00	1 965	2 100	+ 7.0
2.65	2.50	2 250	3 230	+44.0
3.20	2.75	2 990	3 860	+29.0
3.78	3.00	3 840	4 554	+19.0
4.37	3.25	4 770	5 280	+11.0
4.65†	3.35†	5 240	5 590	+ 7.0

† Approximate; taken from curve.

In order to correlate the measurements at the Hydraulic Power Company's dam with those at the weir, observations were taken at each place as nearly cotemporaneously as they could be made by a man going immediately from one to the other. Table No. 1 gives some of the heads actually observed at the weir and dam, together with the discharge over the weir in comparison with the computed discharge over the dam, and the percentage differences.

The crest of the Mount Morris Dam was quite irregular, and, in order to apply weir formulas, an accurate profile was taken and the crest sub-divided into a number of approximately level sections with each section computed separately, advancing by 0.1 ft. up to 10 ft. The flow over the entire dam was obtained by adding together the sums of the several sections at the corresponding heights, and tabulating them. The zero of the gauge was at the level of the lowest section.

The computed discharges, as shown by Columns (3) and (4), are somewhat irregular. This result is due to the disturbing effect of the irregular sections of the crest, the highest point of which was 2 ft. above the lowest.

* For detailed description of this weir see Appendix VII to Annual Report, State Engineer and Surveyor, for 1896, pp. 715–19.

Column (5) shows the percentage variations between the discharges as determined by a sharp-crested weir, up to 5 200 cu. ft. per second, and the discharges computed by the formula cited. These data show at once an error in judgment, excusable only because, previous to the publication of Bazin's paper of 1898, nobody knew how to do better.

In computing the Hudson River gaugings in 1895, the writer used the formula of General Mullins, as fairly applicable to a broad-crested dam like that at Mechanicsville, where Hudson River gaugings have been kept continuously from October, 1887.*

In August, 1898, the writer began an extensive special investigation as to water supply for summit-level canals in the State of New York, for the United States Board of Engineers on Deep Waterways. The magnitude of the commercial interests involved justified a most thorough study, and the work was accordingly carried out on an extended scale. A large collection of new data has been obtained, which, by permission of the Board, the writer has the pleasure of presenting to the Society herein.

At the beginning of the study it was deemed advisable to gauge a number of streams tributary to proposed deep waterways in Central and Eastern New York, not only with reference to extending information as to the low-water flow of the Oswego, Mohawk, Black and Hudson Rivers, but especially to gain more definite information as to the flood flows of these streams and their tributaries, it being recognized clearly that the control of floods in canalized river-beds was a serious feature of the general problem.

To accomplish this, gauging stations were established on Seneca River, at Baldwinsville; Oswego River, at Fulton; Chittenango Creek, at Bridgeport; Oneida Creek, at Kenwood; West Branch of Fish Creek, at McConnellsville; East Branch of Fish Creek, above Point Rock; Salmon River, above High Falls; Mohawk River, at Ridge Mills, Little Falls and Rexford Flats; Nine Mile Creek, below Stittville; Oriskany Creek, at Oriskany and Coleman; Saquoit Creek, at New York Mills; West Canada Creek, at Dolgeville; Garoga Creek, 3 miles above mouth; Cayadutta Creek, below Johnstown, and Schoharie Creek, at Fort Hunter. In addition, gaugings of Hudson River, at Mechanicsville and Fort Edward, and of Schroon River, at Warrensburg,

* For this formula see Mullins' Irrigation Manual, pp. 11-12, 138-139, 171-172. Also see Annual Report, State Engineer and Surveyor, of New York, for 1895, pp. 105-106; and Water Supply and Irrigation Papers of United States Geological Survey, No. 24; Water Resources of the State of New York, Part I, pp. 79-80.

at stations established previously by the writer, were available, as well as of Black River, at Huntingtonville, a suburb of Watertown, at a station established by the Board of Water Commissioners of Watertown.

The foregoing gauging stations are in every case existing dams, either of masonry or timber. Several of them, as at Baldwinsville, Fulton, Little Falls, Middleville, Dolgeville, etc., have extensive power devolopments, with large quantities of water passing through turbine water wheels, for either the whole or a portion of each day. Hardly any two cross-sections are alike, as may be sufficiently appreciated by examining Figs. 8 to 20, although some of them conform generally to certain of Bazin's types, as is shown by the illustrations. Finally, many of them have gross irregularities in the crests, longitudinally, as shown. The method of treatment, in order to obtain approximately correct results, becomes, therefore, a matter of some difficulty. In a few cases, as on Nine Mile Creek, West Canada Creek, etc., where the crests were very irregular, a small amount of work was done in the way of leveling them. Generally, however, the crests were left nearly in the same condition as found, a profile was carefully taken and the crest divided into a series of approximately level sections for computation. A gauging blank was furnished the gauge readers, with columns for entering depth on crest of dam, A. M. and P. M., number of water wheels used, size of same, name of manufacturer and daily run, working head on wheels, readings of head-race and tail-race gauges, and other information necessary for keeping an accurate account of the water passing over the crest in 24 hours, as well as through water wheels for the same period. Gauge readers were employed to take these readings twice each day.

In order to obtain flows through water wheels, recourse was had to records of the test flume of the Holyoke Water Power Company, of Holyoke, Mass., where the principal wheels now in common use in New York State have, at one time or another, been tested. On requesting a record of such tests, as applying to wheels at the several gauging stations, the Holyoke Water Power Company kindly responded that they would furnish the records under the condition that they be not published unless the consent of parties for whom the wheels had been tested were first obtained. This condition being assented to, information was furnished as to tests of the principal wheels in use, giving proportional part of opening of speed gate for various conditions of tests, revolutions

of wheel, quantity of water discharged, power developed, efficiency, etc. From these records, wheel-discharge curves have been prepared for the water wheels in use at each dam. By the use of such curves, derived from actual tests, it is believed that the discharges through turbine water wheels at the various gauging stations have been computed with a very high degree of accuracy. Under these conditions turbine water wheels become in effect efficient water meters. In a few cases, where there were no tests applying, the discharges as per manufacturers' tables have been used. The writer's thanks are due to the Holyoke Water Power Company for the courtesy of furnishing these useful data.

Before describing the method of procedure for obtaining flows over dams at the several gauging stations, we may refer briefly to some of the more salient points of Bazin's papers in *Annales des Ponts et Chaussées.*

In the beginning of his first paper, Bazin remarks that the theory of the weir is the least advanced of all branches of hydraulics. The coefficients used in practice vary between such wide limits that in most cases we are unable to make a rational selection from the many numerical values assigned to them.

The problem, he says, is in fact a complicated one, being connected on the one hand with the theory of flow through orifices and on the other with that of open channels. The value of the coefficients in each case is influenced by many elements. Thus we ought to consider:

(1) The velocity of approach; that is, the velocity with which the up-stream water reaches the weir, the effect of which cannot be neglected in weirs of small height.

(2) The contraction of the vertical section of the stream at the weir, the amount depending upon the height of the weir and the form of the crest.

(3) The lateral contraction which, though unimportant in weirs of great length, seriously modifies the results in shorter weirs.

As a further condition, Bazin points out that when the down-stream channel has a width of the length of the weir, so that the overflowing sheet of water, or nappe, touches at the sides, thus preventing free admission of air under the nappe, there occur special phenomena greatly affecting the flow.*

* Bazin's earlier papers are directed specially to a detailed investigation of these several points. Space will not be taken here to describe his experiments in detail. The original data may be found in the *Annales des Ponts et Chaussées* for the years already cited. A translation of the earlier numbers has also been made by Messrs. Arthur Marichal and John C. Trautwine. Jr., and may be found in the *Proceedings* of the Engineers' Club of Philadelphia for January, 1890; July, 1892; October, 1892, and April, 1896.

Bazin's method of experimentation may be referred to briefly. A standard weir was set up at the head of a long chamber, in which the actual volume passing over was measured a sufficient number of times to give averages, which Bazin considers are accurate to within probably less than 1 per cent. Having established in this way the values of the coefficients for a standard weir, with heads varying from about 0.164 ft. to 1.969 ft., the experiments on weirs of irregular profiles were made by placing each experimental weir below the standard weir, and observing the heads synchronously on each. In these experiments a steady current was established in the channel, and observations of the known volume passing over the standard weir were made, which volume also passed over the weir under investigation, lower down.

If we let H and h denote, respectively, the head upon the standard weir and upon the lower weir, L and l, their corresponding lengths, and M and m, the coefficients of discharge, and then, adopting provisionally Formula (1) for the standard weir—

$$Q = M L H \sqrt{2 g H}; \qquad (1)$$

and similarly for the lower weir

$$Q = m l h \sqrt{2 g h}. \qquad (2)$$

Equating these two values of Q, we have

$$M L H \sqrt{H} \sqrt{2 g} = m l h \sqrt{h} \sqrt{2 g}, \text{ or}$$

$$M L H^{\frac{3}{2}} = m l h^{\frac{3}{2}}$$

from which we deduce the value of m:

$$m = M \left(\frac{L}{l}\right) \times \left(\frac{H}{h}\right)^{\frac{3}{2}}$$

As already stated, Bazin's preliminary gauging operations gave, once for all, the coefficient M for the standard weir for each value of H. The ratio $\frac{L}{l}$, which is very nearly unity, remained constant for all experiments of any one series, and, therefore, we have only to measure the heads H and h in order to obtain the coefficient m.

Fteley and Stearns experimented somewhat on the influence of the height of the weir upon the flow, and probably as interesting a point as any brought out by Bazin's extended discussion is the considerable influence of this element upon the flow. After presenting the detail of experiments on sharp-crested weirs of various heights and for various heads between the limits stated, Bazin gives a table of values of the coefficient m for sharp-crested weirs, ranging in height from

TABLE No. 2.—VALUES OF THE COEFFICIENT m IN THE FORMULA $Q = m\, l\, h \sqrt{2\, g\, h}$, FOR A SHARP-CRESTED WEIR WITHOUT LATERAL CONTRACTION, THE AIR BEING ADMITTED FREELY BENEATH THE OVERFLOWING SHEET OF NAPPE.

Head = h, in feet.	Values of the coefficient m corresponding to the height P of the weir above the bottom of the channel.									Limiting value of m = coefficient u.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Value of P, in feet.	0.656	0.984	1.312	1.640	1.968	2.624	3.280	4.920	6.560	∞
0.164	0.458	0.453	0.451	0.450	0.449	0.449	0.449	0.448	0.448	0.4481
0.197	0.456	0.450	0.447	0.445	0.445	0.444	0.443	0.443	0.443	0.4427
0.230	0.455	0.448	0.445	0.443	0.442	0.441	0.440	0.440	0.439	0.4391
0.262	0.456	0.447	0.443	0.441	0.440	0.438	0.438	0.437	0.437	0.4363
0.295	0.457	0.447	0.442	0.440	0.438	0.436	0.436	0.435	0.434	0.4340
Means..	0.456	0.449	0.446	0.444	0.443	0.442	0.441	0.441	0.440	0.4400
$m\sqrt{2g}$	3.66	3.60	3.58	3.56	3.55	3.54	3.54	3.53	3.53	3.53
0.328	0.459	0.447	0.442	0.439	0.437	0.435	0.434	0.433	0.433	0.4322
0.394	0.462	0.448	0.442	0.438	0.436	0.433	0.432	0.430	0.430	0.4291
0.459	0.466	0.450	0.443	0.438	0.435	0.432	0.430	0.428	0.428	0.4267
0.525	0.471	0.453	0.444	0.438	0.435	0.431	0.429	0.427	0.426	0.4246
0.591	0.475	0.456	0.445	0.439	0.435	0.431	0.428	0.426	0.425	0.4229
Means..	0.467	0.451	0.443	0.438	0.436	0.432	0.431	0.429	0.428	0.4271
$m\sqrt{2g}$	3.74	3.62	3.56	3.52	3.50	3.47	3.46	3.44	3.44	3.43
0.656	0.480	0.459	0.447	0.440	0.436	0.431	0.428	0.425	0.423	0.4215
0.722	0.484	0.462	0.449	0.442	0.437	0.431	0.428	0.424	0.423	0.4203
0.787	0.488	0.465	0.452	0.444	0.438	0.432	0.428	0.424	0.422	0.4194
0.853	0.492	0.468	0.455	0.446	0.440	0.432	0.429	0.424	0.422	0.4187
0.919	0.496	0.472	0.457	0.448	0.441	0.433	0.429	0.424	0.422	0.4181
Means..	0.488	0.465	0.452	0.444	0.438	0.432	0.428	0.424	0.422	0.4196
$m\sqrt{2g}$	3.92	3.73	3.63	3.56	3.52	3.46	3.44	3.40	3.39	3.37
0.984	0.500	0.475	0.460	0.450	0.443	0.434	0.430	0.424	0.421	0.4147
1.050	0.500	0.478	0.462	0.452	0.444	0.436	0.430	0.424	0.421	0.4168
1.116	0.500	0.481	0.464	0.454	0.446	0.437	0.431	0.424	0.421	0.4162
1.181	0.500	0.483	0.467	0.456	0.448	0.438	0.432	0.424	0.421	0.4156
1.247	0.500	0.486	0.469	0.458	0.449	0.439	0.432	0.424	0.421	0.4150
Means..	0.500	0.481	0.464	0.454	0.446	0.437	0.431	0.424	0.421	0.4162
$m\sqrt{2g}$	4.01	3.86	3.73	3.64	3.58	3.50	3.46	3.40	3.38	3.34
1.312	0.500	0.489	0.472	0.459	0.451	0.440	0.433	0.424	0.421	0.4144
1.378	0.500	0.491	0.474	0.461	0.452	0.441	0.434	0.425	0.421	0.4139
1.444	0.500	0.494	0.476	0.463	0.454	0.442	0.435	0.425	0.421	0.4134
1.509	0.500	0.496	0.478	0.465	0.456	0.443	0.435	0.425	0.421	0.4128
1.575	0.500	0.496	0.480	0.467	0.457	0.444	0.436	0.425	0.421	0.4122
Means..	0.500	0.493	0.476	0.463	0.454	0.442	0.435	0.425	0.421	0.4133
$m\sqrt{2g}$	4.01	3.96	3.82	3.72	3.64	3.55	3.49	3.41	3.38	3.32
1.640	0.500	0.496	0.482	0.468	0.459	0.445	0.437	0.426	0.421	0.4118
1.706	0.500	0.496	0.483	0.470	0.460	0.446	0.438	0.426	0.421	0.4112
1.772	0.500	0.496	0.485	0.472	0.461	0.447	0.438	0.426	0.421	0.4107
1.837	0.500	0.496	0.487	0.473	0.463	0.448	0.439	0.427	0.421	0.4101
1.903	0.500	0.496	0.489	0.475	0.464	0.449	0.440	0.427	0.421	0.4096
1.969	0.500	0.496	0.490	0.476	0.466	0.451	0.441	0.427	0.421	0.4092
Means..	0.500	0.496	0.486	0.472	0.462	0.448	0.436	0.427	0.421	0.4104
$m\sqrt{2g}$	4.01	3.98	3.90	3.79	3.71	3.60	3.50	3.42	3.38	3.29

0.656 ft. to 6.56 ft. (0.2 to 2.0 m.). Column (11) of Table No. 2 gives the limiting value of m, which equals the coefficient u of Bazin's formulas, and which represents the value of m for a weir of infinite height, or of such a height that the height of the weir above the bottom of the channel has no further effect upon the flow. As will be seen by examining this table, the influence beyond 6.56 ft. is only slight.

The formula for a sharp-crested weir without end contractions, as ordinarily used in the United States, is that of Mr. Francis, namely:

$$Q = Clh^{\frac{3}{2}}.$$

In this formula $C = m\sqrt{2g}$ of Bazin's formula; hence, by making a simple multiplication we may express Bazin's values of m in terms of the formula in common use in the United States. The values of $m\sqrt{2g}$ for different heads on the crest and for heights of weir varying from 0.656 ft. to 6.56 ft., and also for the limiting value of m may be seen as carried out in Table No. 2.

In regard to the accuracy of the coefficients given by this table, Bazin remarks that, except in the unusual case of a very low weir, which should always be avoided, it will give the coefficient m within 1%, provided, however, that the arrangements of his standard weir be exactly reproduced. It is also pointed out as especially important that the admission of air behind the falling sheet be perfectly assured; otherwise m may vary within much wider limits.

Description of Bazin's Experiments on Weirs of Irregular Cross-Section.

The following statements have been condensed from Bazin's paper in *Annales des Ponts et Chaussées*, for 1898.

We will now consider weirs in which the back and front faces, instead of being vertical, have a batter of greater or less inclination. The conditions of discharge will be found to be greatly modified. The batter on the up-stream side tends to diminish the contraction in passing over the crest, and hence to increase the discharge. The influence of the down-stream batter, on the other hand, is not always constant, but varies according to the degree of inclination. If the down-stream face is not far from vertical, the nappe adheres to it for small discharges, but becomes detached at a certain head and then follows the condition described as wetted underneath, analogous to

that which we have studied for square timber weirs. If, on the contrary, the batter is nearly horizontal, the nappe does not detach itself, but remains in contact with the face of the weir under all heads. The discharge may, however, vary greatly according to the degree of inclination of the face. On the other hand, the influence of the width of crest is considerable, as we have seen in the case of weirs formed of square timbers. Hence, a weir with a wide crest and inclined faces may present a large variety of results, each type having, so to speak, its own special scale of coefficients. Such a study, to be complete, must include a very considerable number of particular cases. Without embracing all possible cases, the experiments made have been somewhat numerous. They include weirs with different degrees of batter on the front and back faces, and having sharp crests, on the one hand, and on the other, crests 0.10, 0.20 and 0.40 m. in width (0.328, 0.656 and 1.312 ft., respectively).

Some additional experiments have been made on weirs having crests joined to the inclined faces by curved surfaces, and, finally, weirs having completely curved profiles have been experimented upon.

We may divide the numerous series of experiments into five groups, namely:

(1) Weirs so nearly vertical on the down-stream side that the nappe remains detached.

(2) Weirs having the up-stream face vertical, or nearly vertical, but with a batter on the down-stream face so nearly horizontal that the water always remains in contact.

(3) Weirs both faces of which are at an inclination differing from the horizontal by less than 45 degrees.

(4) Weirs in which the crest is joined to the inclined faces by curved surfaces.

(5) Weirs having completely curved profiles.

First Group.—Weirs Having the Down-Stream Face Vertical or Nearly Vertical.

This group includes ten series of experiments, the results of which differ notably, according to inclination of batter and width of crest. The values of the coefficient $\frac{m}{m'}$, which have been obtained for sharp-crested weirs, have been compared with those corresponding to nappes

wetted underneath with sharp-crested weirs. Similarly, we may compare conveniently the values obtained for weirs with crests 0.10 and 0.20 m. wide (0.328 and 0.656 ft., respectively), with those for flat-crested beam weirs of the same width in the table.*

The coefficients have been made to follow a uniform law, by plotting the immediate results of the experiments in such a manner that the head h represents the abscissa and the ratio $\frac{m}{m'}$ the ordinate of a point representing one of the experiments. A mean curve has been drawn by the aid of these points, and from this curve, drawn on a large scale, values of the ratio $\frac{m}{m'}$ have been taken, corresponding, roundly, to abscissas of 0.10 m., 0.15 m., etc. (0.328 ft., 0.492 ft., etc.). All the experiments terminate with nappes wetted underneath, but the discharge for these, as well as for depressed and adhering nappes, is shown in the table, the nature of the nappe in each case being indicated in the proper column. If we consider, first, sharp-crested weirs, we perceive that the appearance of the wetted nappe is preceded by the depressed nappe, imprisoning air between its under surface and the body of the weir, excepting in the case of a weir with a face batter of 3 : 2, which permits the formation of an adhering nappe. When once the wetted nappe is established, its coefficient does not differ greatly from that for a sharp-crested weir, excepting in the cases where the batter on the back is 3 : 1 and 3 : 2, when it is greater. For weirs with flat crests 0.10 m. (0.328 ft.) wide, the adhering form of nappe appears when the up-stream face has a batter of 3 : 1 or 3 : 2. The ratio $\frac{m}{m'}$ is also modified, however, by adherence of the water to the flat crest. At the moment when this adhesion ceases, the coefficient diminishes suddenly about 10 per cent. This has taken place in the two series, Nos. 133 and 134, where the up-stream face is vertical. In the other series, the head has not been sufficiently large to detach the nappe, and the coefficient remains, in these experiments, greater in value than it would be for a beam weir in which the nappe has become detached before the given head is attained.

Only one series of experiments has been made on weirs with crest 0.20 m. (0.656 ft.) wide, the down-stream face being vertical and that on the up-stream side having a batter of 1 : 2. In accordance with the

* See p. 159, *Annales des Ponts et Chaussées, 2d Trimestre*, 1898.

increased width of crest the wetted nappe does not appear until quite late, and with a coefficient notably diminished. It is clear that more extended experiments would have led in all cases to results differing in a similar manner from those for crests 0.10 m. in width. Without going farther we may say that, on a weir having the down-stream face nearly vertical, the wetted form of nappe always appears when a certain head has been attained. This limiting head varies with the inclination of the front and back faces, and is never the same as that which corresponds to detachment of the nappe from the flat crest, which also influences the value of the coefficient. Each type of weir requires a special study, and, in view of the complexity of conditions involved, it is impossible to establish a general formula for the discharge coefficient.

Second Group.—Face of the Weir on the Up-Stream Side Vertical or Nearly Vertical.

Let us now consider the case where, in contradistinction to the first group, the back is nearly vertical and the down-stream face has considerable batter. For sharp-crested weirs with the up-stream face vertical the ratio $\frac{m}{m'}$ is nearly constant for each series, the mean values being 1.13, 1.03, 0.90 and 0.84 for batters on the down-stream face of 1 : 1, 1 : 2, 1 : 5 and 1 : 10, respectively. Where the up-stream face has a batter of 3 : 1 or 2 : 1, it produces an increase, in the value of the coefficient, of a few per cent. Turning to the experiments on weirs with crests 0.10, 0.20 and 0.40 m. (0.328, 0.656 and 1.312 ft., respectively) in width, it may be seen that the coefficients increase with the head, Series No. 143 only showing, for the final values, a rapid diminution, proving that the nappe has become detached. It is necessary, in fact, in order that the nappe shall become detached, that the head shall be greater in proportion as the width of the crest becomes greater and the slope of the down-stream face more gentle. This limit has not, in general, been reached in the experiments, and the nappes remain attached to the crests, excepting for Series No. 143 (batter on the down-stream side 1 : 1), where the detachment of the nappe leads to a diminution of the coefficient ratio from 1.21 to 1.14. The next series, No. 144, indicates also a slight tendency of the ratio $\frac{m}{m'}$ to decrease, the batter here being 1 : 2. This tendency entirely

disappears where the slope of the down-stream face is not greater than 1 : 3 or 1 : 4. Omitting the results for the relatively small heads of 0.10 m. (0.328 ft.) or less, where there are some irregularities, the increments of the coefficient ratio $\frac{m}{m'}$ for heads from 0.10 m. (0.328 ft.) to the limits of the experiments, are given for each series, in comparison with the results for a weir with a crest 0.10 m. (0.328 ft.) in width, Series Nos. 133 and 134, and with those for flat-crested weirs, 0.40 m. (1.312 ft.) and 2.00 m. (6.56 ft.) in width, Series Nos. 113 and 115. Such comparison shows, first, that for the same width of crest, $\frac{m}{m'}$ diminishes when the inclination of the down-stream face is gradually diminished below 45°; second, that, other things being equal, that is to say, for the same batters on the two faces in each case, the ratio $\frac{m}{m'}$ diminishes when the width of crest is increased.

Third Group.—Batter of the Up-Stream and Down-Stream Faces Very Gentle, not Exceeding 45 Degrees.

Weirs encountered in practice do not often have nearly vertical faces like those we have been considering, but have slopes inclined 45°, or more, from the vertical. Such weirs have been made the subject of a series of experiments, in which slopes of 1 : 1 and 1 : 2 on the up-stream side have been combined with slopes of 1 : 1, 1 : 2 and 1 : 5 on the down-stream side for three different widths of crest.

In most cases the ratio $\frac{m}{m'}$ increases with the head, but, in order to investigate more fully the changes in value of $\frac{m}{m'}$, it is necessary to consider separately the case of sharp-crested weirs as distinguished from those with wide crests.

Sharp-Crested Weirs.

Batter 1 : 1 on Down-Stream Side.—The coefficient ratio $\frac{m}{m'}$, decreases as the head h increases from above 1.20 for very slight heads to 1.11 or 1.12 for the greatest heads used. Its value is sensibly the same for the two batters of 1 : 1 and 1 : 2 on the back. The rate of decrease is not uniform, being very gradual to $h = 0.30$ m. (0.984 ft.), beyond

which it changes rapidly, without doubt due to the detachment of the nappe.

Batter 1 : 2 on Down-Stream Side.—Instead of decreasing as the head increases, $\frac{m}{m'}$, increases slowly in value between the limits 1.10 and 1.13, its value being nearly the same for the two batters of 1 : 1 and 1 : 2 on the back.

Batter 1 : 5 on Down-Stream Side.—In this case the coefficient ratio is nearly independent of h, decreasing from 1.015 to 1.000 for a batter of 1 : 1 on the back, and from 1.045 to 1.035 for a batter of 1 : 2 on the back.

Weirs Having Crests 0.10 and 0.20 M. (0.328 and 0.656 Ft.) in Width.

The coefficient always increases with the head, but the limits between which this increase takes place differ in each case. If it were possible to increase the head indefinitely, and at the same time the height of the weir, the conditions of discharge would approach progressively those for a sharp-crested weir, the width of the crest becoming more and more negligible, relative to the general dimensions of the dam. The series of coefficients relating to crests of a given width cannot be, with certainty, extended beyond the experimental limits between which they were obtained. If, however, the curves representing the coefficients were prolonged sufficiently, they would converge toward those which correspond to a sharp-crested weir with vertical faces. The batter on the back determines the direction of the filaments which constitute the inferior surface of the nappe, and influences thus the contraction at the inner edge of the crest, and hence also the discharge. The batter on the down-stream side, on the other hand, affects the pressure underneath the nappe.

When width of crest is not negligible, the inclination of the down-stream face determines the limiting head at which the nappe detaches itself from the crest, and at which point the conditions of discharge change suddenly. This limit depends also, in a certain measure, on the velocity of approach, or, which amounts to the same thing, on the ratio of the head h to the height p of the weir. A complete formula should include, in addition to the slopes of the two faces, the two ratios, $\frac{h}{c}$ and $\frac{h}{p}$, where c is the width of the flat portion of the crest. Such a formula would be excessively complicated.

Fourth Group.—The Two Faces United by a Curved Surface at the Crest of the Weir.

Seven types of weirs were experimented on, with a view to determining the effect on the discharge of joining the two faces of the weir by a curved surface, the slopes used being, on the up-stream side, very nearly vertical (5 : 1), and on the down-stream side from 1 : 3 to 1 : 5. The up-stream edge of the crest being rounded to an arc of 0.05, 0.10 or 0.20 m. (0.164, 0.328 or 0.656 ft.) radius.

Types Nos. 1 and 2 differ only in regard to the radius of curvature of the back edge of the crest, this being 0.05 m. (0.164 ft.) for the first, and 0.10 m. (0.328 ft.) for the second. The radius of 0.10 m. increases the discharge slightly more than that of 0.05 m., though the difference is unimportant, but the values of $\frac{m}{m'}$ in both cases surpass considerably those which have been obtained for similar weirs with the two faces united by flat crests 0.10 and 0.20 m. (0.328 and 0.656 ft.) in width. Series Nos. 145 and 153.

Types Nos. 3 and 4 are similar to Types Nos. 1 and 2, except as regards the inclination on the down-stream side, which has been reduced from 1 : 3 to 1 : 5. This conduces to equalize the values of m, which differ relatively little from each other, although the values are sensibly less at the same heads than those for Types Nos. 1 and 2, and yet somewhat greater than for weirs with flat crests, 0.10 and 0.20 m. (0.328 and 0.656 ft.) in width. Series Nos. 145, 155 and 156.

Types Nos. 5 and 6 differ only by the radius of curvature at the back, which is 0.10 and 0.20 m., respectively, the crest being wider than in the preceding cases. The value of m is somewhat less than before, not differing greatly from that which corresponds to Types Nos. 3 and 4.

The width of crest is increased still further in Type No. 7, the length of the rectilinear section between the origins of the two curved surfaces being 0.20 m. (0.656 ft.). This modification produces a sensible diminution in the value of m.

Fifth Group.—Weirs with Completely Curved Profiles.

We turn, finally, to the consideration of weirs having completely curved profiles. The coefficient m then attains exceptionally high values. Types Nos. 1 and 2, having vertical down-stream faces,

permit of the formation of nappes wetted underneath. The corresponding coefficients are much higher than for the analogous case with a sharp-crested weir.*

Types Nos. 3 and 4 have crests which differ only in the radii of curvature of the curved surfaces, being 0.05 and 0.08 m. (0.164 and 0.2624 ft.), respectively, for Type No. 3, and 0.10 and 0.12 m. (0.328 and 0.3936 ft.), respectively, for Type No. 4. The crest, in this latter case, is much larger and the coefficient m is, as a result, notably less for heads up to 0.30 m. (0.984 ft.), but it is not the same beyond this point, for the concave form of Type No. 3 tends to produce detachment of the nappe, and the coefficient for this type continues to diminish from this point, becoming less than for Type No. 4, in which the coefficient increases for all heads within the limits of the experiments.

The results of Bazin's experiments on weirs of irregular profiles, except a few of the series which have been omitted, will be found on the pages indicated in the following list. The value of $m \sqrt{2\,g}$ is given in every case as derived from Bazin's tabulated value of m:

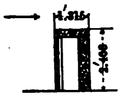
BAZIN'S EXPERIMENTS.

Series No.	Page.	Series No.	Page.	Series No.	Page.
113	242	140	246	163	253
114	280†	141	247	164	253
115	282†	142	247	165	253
116	281†	143	247	166	254
117	283†	144	248	167	254
125	242	145	248	168	254
126	242	146	248	170	255
127	243	147	249	172	255
128	243	149	249	173	279†
129	243	150	249	175	255
130	272†	151	250	176	256
131	244	153	250	177	256
132	244	154	250	178	278†
133	244	156	251	179	256
134	245	157	251	181	257
135	273†	158	251	182	257
136	245	159	252	188	257
137	245	160	252	189	258
138	246	161	252	193	258
139	246	162	277†		

* For the type-forms under this group, see Bazin's paper.
† With diagram of similar Cornell experiment.

BAZIN'S

SERIES NO. 113



No. of Experiment.	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' Formula.	
1	0.206	0.3294	2.64	
2	0.289	0.3313	2.66	
3	0.363	0.3307	2.66	
4	0.443	0.3302	2.65	
5	0.518	0.3321	2.66	
6	0.592	0.3348	2.64	
7	0.667	0.3378	2.71	
8	0.736	0.3427	2.75	
9	0.805	0.3468	2.78	
10	0.863	0.3494	2.80	
11	0.936	0.3547	2.85	
12	0.989	0.3593	2.88	
13	1.035	0.3628	2.91	
14	1.076	0.3660	2.94	
15	1.114	0.3682	2.95	
16	1.159	0.3734	3.00	
17	1.197	0.3758	3.01	
18	1.262	0.3821	3.06	
19	1.330	0.3877	3.11	

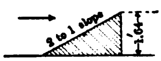
BAZIN'S

SERIES NO. 125

1	0.318	0.4516	3.62	— Nappe depressed.
2	0.383	0.4530	3.63	
3	0.439	0.4604	3.69	
4	0.492	0.4715	3.79	
5	0.532	0.4978	4.00	
6	0.569	0.5258	4.22	— Nappe wetted underneath.
7	0.633	0.5178	4.16	
8	0.691	0.5187	4.16	
9	0.752	0.5100	4.09	
10	0.818	0.5082	4.09	
11	0.878	0.5047	4.05	
12	0.950	0.4896	4.00	
13	1.006	0.4996	4.01	
14	1.077	0.4954	3.97	
15	1.140	0.4905	3.93	
16	1.190	0.4919	3.95	
17	1.266	0.4892	3.92	
18	1.328	0.4883	3.91	
19	1.395	0.4863	3.90	

BAZIN'S

SERIES NO. 126



1	0.295	0.5015	4.02	— Nappe depressed.
2	0.360	0.5028	4.03	
3	0.413	0.5024	4.03	
4	0.467	0.5138	4.12	
5	0.520	0.5205	4.17	
6	0.567	0.5332	4.27	— Nappe wetted underneath.
7	0.627	0.5256	4.21	
8	0.684	0.5238	4.20	
9	0.740	0.5251	4.21	
10	0.804	0.5244	4.20	
11	0.860	0.5234	4.20	
12	0.920	0.5233	4.20	
13	0.961	0.5203	4.17	
14	1.040	0.5228	4.19	
15	1.105	0.5175	4.15	
16	1.164	0.5197	4.17	
17	1.222	0.5170	4.15	
18	1.284	0.5187	4.16	
19	1.351	0.5142	4.12	

BAZIN'S

SERIES NO. 127.*

No. of experiment.	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula.	
1	0.324	0.4326	3.47	Nappe depressed
2	0.389	0.4384	3.51	
3	0.449	0.4419	3.54	
4	0.509	0.4454	3.57	
5	0.561	0.4606	3.69	
6	0.608	0.4728	3.80	
7	0.653	0.4933	3.95	
8	0.697	0.5078	4.07	Nappe wetted
9	0.764	0.4989	4.00	underneath.
10	0.831	0.4935	3.96	
11	0.896	0.4895	3.93	
12	0.961	0.4885	3.92	
13	1.033	0.4812	3.86	
14	1.093	0.4823	3.87	
15	1.166	0.4720	3.79*	
16	1.228	0.4746	3.81	
17	1.292	0.4729	3.80	
18	1.360	0.4726	3.79	
19	1.424	0.4715	3.78	

*See Bazin's observations relative to the behavior of the nappe for Series Nos. 127 and 128.

BAZIN'S

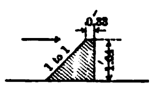
SERIES NO. 128.*

1	0.295	0.5015	4.02	Nappe adhering.
2	0.353	0.5104	4.09	
3	0.407	0.5090	4.09	
4	0.464	0.5107	4.10	
5	0.526	0.5068	4.07	
6	0.580	0.5159	4.14	
7	0.626	0.5183	4.16	
8	0.692	0.5179	4.15	
9	0.737	0.5156	4.14	
10	0.805	0.5178	4.15	
11	0.848	0.5133	4.12	
12	0.890	0.4909	3.94	Nappe wetted
13	0.925	0.4955	3.97	underneath.
14	0.967	0.4849	3.89	
15	1.023	0.4834	3.87	
16	1.096	0.4820	3.87	
17	1.166	0.4766	3.83	
18	1.230	0.4758	3.82	
19	1.294	0.4725	3.79	
20	1.360	0.4738	3.80	
21	1.430	0.4696	3.77	

*See Bazin's observations relative to the behavior of the nappe for Series Nos. 127 and 128.

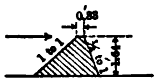
BAZIN'S

SERIES NO. 129.



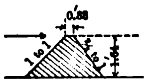
1	0.343	0.4064	3.26	Nappe depressed
2	0.399	0.4225	3.39	
3	0.456	0.4343	3.48	
4	0.511	0.4463	3.57	
5	0.565	0.4577	3.67	
6	0.612	0.4722	3.79	Nappe wetted
7	0.665	0.4778	3.83	underneath and
8	0.726	0.4838	3.88	attached to flat
9	0.774	0.4952	3.97	crest.
10	0.830	0.4982	4.00	
11	0.885	0.5029	4.04	
12	0.943	0.5079	4.08	
13	0.995	0.5116	4.11	
14	1.047	0.5155	4.14	
15	1.102	0.5187	4.16	
16	1.163	0.5174	4.15	
17	1.219	0.5236	4.21	
18	1.266	0.5263	4.23	
19	1.327	0.5276	4.23	

BAZIN'S
SERIES NO. 131



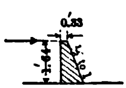
No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.	
1	0.347	0.3969	3.20	Nappe depressed.
2	0.336	0.4140	3.31	Nappe adhering.
3	0.395	0.4298	3.45	
4	0.450	0.4326	3.47	
5	0.504	0.4443	3.56	
6	0.564	0.4611	3.70	
7	0.610	0.4755	3.82	
8	0.667	0.4770	3.83	
9	0.720	0.4888	3.91	
10	0.773	0.4937	3.96	Nappe wetted under-
11	0.835	0.4948	3.97	neath and attached
12	0.889	0.4997	4.01	to flat crest.
13	0.943	0.5048	4.05	
14	0.998	0.5090	4.08	
15	1.046	0.5128	4.12	
16	1.098	0.5172	4.15	
17	1.161	0.5167	4.15	
18	1.213	0.5226	4.20	
19	1.264	0.5249	4.21	
20	1.333	0.5242	4.21	
21	1.394	0.5234	4.20	

BAZIN'S
SERIES NO. 132



1	0.337	0.4128	3.31	Nappe adhering.
2	0.398	0.4251	3.41	
3	0.453	0.4344	3.48	
4	0.509	0.4485	3.60	
5	0.563	0.4609	3.70	
6	0.619	0.4663	3.74	
7	0.665	0.4788	3.84	
8	0.719	0.4876	3.92	
9	0.779	0.4921	3.95	
10	0.826	0.5029	4.03	
11	0.874	0.5108	4.10	
12	0.926	0.5180	4.16	
13	0.974	0.5237	4.21	
14	1.047	0.5136	4.12	Nappe wetted under-
15	1.099	0.5174	4.15	neath and attached
16	1.165	0.5177	4.15	to flat crest.
17	1.212	0.5225	4.19	
18	1.271	0.5238	4.21	
19	1.328	0.5260	4.22	

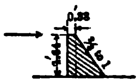
BAZIN'S
SERIES NO. 133



1	0.349	0.3846	3.09	Nappe depressed.
2	0.406	0.4028	3.23	
3	0.400	0.4127	3.31	Nappe adhering.
4	0.456	0.4266	3.42	
5	0.506	0.4445	3.57	
6	0.565	0.4540	3.64	
7	0.610	0.4692	3.76	
8	0.666	0.4775	3.83	
9	0.707	0.4922	3.95	
10	0.767	0.4979	4.00	
11	0.816	0.5079	4.07	
12	0.870	0.5128	4.11	Nappe wetted under-
13	0.928	0.5151	4.13	neath and attached
14	0.978	0.5208	4.17	to flat crest.
15	1.033	0.5224	4.19	
16	1.095	0.5243	4.21	
17	1.205	0.4889	3.92	Nappe wetted under-
18	1.283	0.4794	3.84	neath but detached
19	1.276	0.4842	3.88	from flat crest.
20	1.346	0.4793	3.84	
21	1.415	0.4783	3.83	

BAZIN'S

SERIES No. 134



No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.	
1	0.350	0.3880	3.11	Nappe adhering.
2	0.405	0.4090	3.28	
3	0.458	0.4228	3.39	
4	0.513	0.4378	3.51	
5	0.567	0.4515	3.62	
6	0.615	0.4633	3.71	
7	0.664	0.4751	3.81	
8	0.714	0.4867	3.91	
9	0.767	0.4970	3.99	
10	0.815	0.5061	4.06	
11	0.866	0.5186	4.16	
12	0.909	0.5280	4.24	
13	0.964	0.5334	4.28	
14	1.011	0.5380	4.32	
15	1.060	0.5427	4.36	
16	1.118	0.5463	4.38	
17	1.253	0.4957	3.98	Nappe wetted underneath but detached from flat crest.
18	1.348	0.4804	3.85	
19	1.574	0.4764	3.82	
20	1.489	0.4712	3.78	

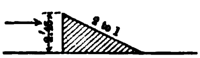
BAZIN'S

SERIES No. 136

No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.183	0.4865	3.90
2	0.244	0.4815	3.86
3	0.304	0.4803	3.85
4	0.364	0.4814	3.86
5	0.424	0.4831	3.88
6	0.484	0.4820	3.87
7	0.542	0.4842	3.88
8	0.597	0.4854	3.89
9	0.658	0.4874	3.91
10	0.718	0.4886	3.92
11	0.776	0.4900	3.93
12	0.830	0.4948	3.97
13	0.887	0.4941	3.96
14	0.953	0.4962	3.98
15	1.010	0.4950	3.97
16	1.068	0.4979	4.00
17	1.122	0.4975	3.99
18	1.179	0.4997	4.01
19	1.244	0.4993	4.01
20	1.299	0.5006	4.01
21	1.361	0.5023	4.03

BAZIN'S

SERIES No. 137



No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.268	0.4324	3.47
2	0.339	0.4310	3.46
3	0.391	0.4359	3.50
4	0.451	0.4323	3.47
5	0.513	0.4400	3.53
6	0.578	0.4377	3.51
7	0.637	0.4378	3.51
8	0.700	0.4434	3.56
9	0.765	0.4437	3.56
10	0.822	0.4441	3.56
11	0.887	0.4445	3.56
12	0.946	0.4513	3.62
13	1.012	0.4476	3.59
14	1.078	0.4505	3.61
15	1.142	0.4490	3.60
16	1.201	0.4517	3.62
17	1.262	0.4520	3.62
18	1.322	0.4543	3.64

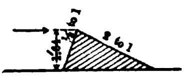
BAZIN'S

SERIES No. 138.

No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.194	0.4450	3.57
2	0.263	0.4357	3.50
3	0.327	0.4344	3.48
4	0.391	0.4356	3.50
5	0.447	0.4429	3.56
6	0.510	0.4524	3.63
7	0.571	0.4511	3.62
8	0.626	0.4622	3.71
9	0.686	0.4565	3.66
10	0.745	0.4597	3.69
11	0.807	0.4611	3.70
12	0.873	0.4637	3.72
13	0.927	0.4643	3.72
14	0.992	0.4683	3.76
15	1.045	0.4743	3.80
16	1.110	0.4707	3.78
17	1.176	0.4716	3.79
18	1.233	0.4746	3.81
19	1.289	0.4758	3.82
20	1.355	0.4758	3.82
21	1.427	0.4778	3.83

BAZIN'S

SERIES No. 139.



1	0.190	0.4561	3.66
2	0.253	0.4585	3.68
3	0.312	0.4641	3.72
4	0.375	0.4570	3.66
5	0.434	0.4651	3.73
6	0.500	0.4643	3.72
7	0.552	0.4706	3.78
8	0.615	0.4693	3.76
9	0.667	0.4764	3.82
10	0.733	0.4725	3.79
11	0.798	0.4734	3.80
12	0.852	0.4789	3.84
13	0.915	0.4808	3.86
14	0.969	0.4821	3.87
15	1.023	0.4883	3.92
16	1.092	0.4856	3.90
17	1.151	0.4868	3.90
18	1.210	0.4909	3.94
19	1.258	0.4927	3.95
20	1.326	0.4904	3.93
21	1.394	0.4898	3.93

BAZIN'S

SERIES No. 140.

1	0.191	0.4696	3.77
2	0.252	0.4666	3.75
3	0.308	0.4677	3.75
4	0.371	0.4620	3.71
5	0.436	0.4699	3.77
6	0.487	0.4666	3.75
7	0.549	0.4747	3.81
8	0.604	0.4761	3.82
9	0.664	0.4783	3.83
10	0.719	0.4792	3.84
11	0.785	0.4842	3.88
12	0.837	0.4843	3.88
13	0.905	0.4889	3.92
14	0.961	0.4861	3.90
15	1.023	0.4920	3.95
16	1.080	0.4905	3.93
17	1.143	0.4951	3.97
18	1.195	0.4930	3.96
19	1.254	0.4951	3.97
20	1.316	0.4978	3.99
21	1.375	0.5000	4.01

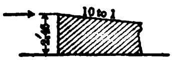
BAZIN'S

SERIES No. 141.

No. of experiment.	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula.
1	0.214	0.3771	3.02
2	0.281	0.3854	3.09
3	0.355	0.3831	3.07
4	0.425	0.3790	3.04
5	0.489	0.3836	3.06
6	0.561	0.3846	3.08
7	0.624	0.3957	3.17
8	0.692	0.3882	3.11
9	0.758	0.3901	3.12
10	0.822	0.3933	3.15
11	0.888	0.3960	3.17
12	0.956	0.3977	3.19
13	1.029	0.3978	3.19
14	1.113	0.3955	3.17
15	1.165	0.3996	3.21
16	1.237	0.3987	3.20
17	1.298	0.4016	3.22
18	1.369	0.4016	3.22
19	1.431	0.4047	3.24
20	1.463	0.4051	3.25

BAZIN'S

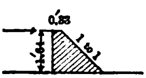
SERIES No. 142.



1	0.300	0.3537	2.83
2	0.369	0.3624	2.90
3	0.447	0.3582	2.87
4	0.509	0.3566	2.86
5	0.591	0.3594	2.88
6	0.666	0.3572	2.86
7	0.727	0.3639	2.92
8	0.795	0.3660	2.94
9	0.861	0.3661	2.94
10	0.934	0.3681	2.95
11	1.007	0.3677	2.95
12	1.079	0.3710	2.98
13	1.149	0.3716	2.98
14	1.222	0.3726	2.99
15	1.285	0.3738	3.00
16	1.362	0.3742	3.00
17	1.430	0.3752	3.01

BAZIN'S

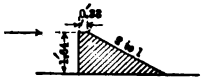
SERIES No. 143.



1	0.352	0.3886	3.11	Nappe adhering and
2	0.415	0.3985	3.19	attached to flat crest.
3	0.462	0.4250	3.41	
4	0.513	0.4386	3.52	
5	0.567	0.4515	3.62	
6	0.619	0.4608	3.70	
7	0.675	0.4704	3.77	
8	0.725	0.4803	3.85	
9	0.771	0.4938	3.96	
10	0.821	0.5059	4.06	
11	0.874	0.5135	4.12	
12	0.920	0.5208	4.18	
13	0.972	0.5283	4.24	
14	1.022	0.5321	4.27	
15	1.072	0.5404	4.34	
16	1.126	0.5435	4.36	
17	1.172	0.5489	4.40	
18	1.227	0.5514	4.42	
19	1.331	0.5250	4.21	Nappe adhering but
20	1.369	0.5229	4.20	detached from flat crest.

BAZIN'S

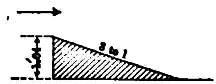
SERIES NO. 144



No. of experi- ment	h=depth on crest, in feet.	m=coeffi- cient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula.
1	0.358	0.3806	3.05
2	0.419	0.3954	3.17
3	0.475	0.4061	3.27
4	0.533	0.4205	3.37
5	0.592	0.4255	3.41
6	0.648	0.4372	3.50
7	0.700	0.4433	3.55
8	0.757	0.4531	3.63
9	0.810	0.4588	3.68
10	0.865	0.4679	3.75
11	0.919	0.4714	3.78
12	0.978	0.4777	3.83
13	1.034	0.4774	3.83
14	1.104	0.4759	3.82
15	1.164	0.4766	3.82
16	1.232	0.4769	3.83
17	1.286	0.4784	3.84
18	1.351	0.4788	3.84
19	1.411	0.4811	3.86

BAZIN'S

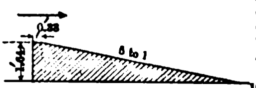
SERIES NO. 145



1	0.369	0.3766	3.02
2	0.424	0.3870	3.10
3	0.479	0.3971	3.18
4	0.547	0.4059	3.25
5	0.598	0.4177	3.35
6	0.658	0.4218	3.38
7	0.720	0.4262	3.42
8	0.781	0.4322	3.47
9	0.836	0.4358	3.49
10	0.902	0.4409	3.53
11	0.962	0.4409	3.53
12	1.032	0.4425	3.53
13	1.087	0 4461	3.58
14	1.152	0 4474	3.58
15	1.210	0.4505	3.61
16	1.274	0.4502	3.61
17	1.334	0.4515	3.62
18	1.396	0.4538	3.64
19	1.467	0.4544	3.64

BAZIN'S

SERIES NO. 146



1	0.367	0.3632	2.91
2	0.437	0.3698	2.97
3	0.492	0.3837	3.08
4	0.567	0.3804	3.05
5	0.621	0.3923	3.14
6	0.696	0.3885	3.11
7	0.749	0.3987	3.20
8	0.817	0.3988	3.20
9	0.883	0.4021	3.22
10	0.951	0.4024	3.22
11	1.012	0.4080	3.27
12	1.065	0.4073	3.26
13	1.141	0.4119	3.30
14	1.213	0.4124	3.30
15	1.279	0.4153	3.33
16	1.344	0.4169	3.34
17	1.405	0.4190	3.36
18	1.473	0.4211	3.38
19	1.532	0.4233	3.39

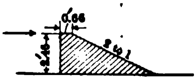
BAZIN'S

SERIES NO. 147

No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge	$C=m\sqrt{2g}$=coefficient for Francis' formula
1	0.231	0.3429	2.75
2	0.306	0.3552	2.85
3	0.373	0.3566	2.86
4	0.433	0.3696	2.97
5	0.503	0.3759	3.02
6	0.569	0.3907	3.13
7	0.637	0.3999	3.20
8	0.681	0.4045	3.24
9	0.734	0.4170	3.34
10	0.797	0.4238	3.40
11	0.845	0.4299	3.44
12	0.898	0.4409	3.53
13	0.953	0.4453	3.57
14	1.015	0.4536	3.63
15	1.063	0.4580	3.67
16	1.115	0.4653	3.73
17	1.166	0.4723	3.79
18	1.217	0.4776	3.83
19	1.265	0.4803	3.85
20	1.332	0.4854	3.89
21	1.394	0.4962	3.98

BAZIN'S

SERIES NO. 149



No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge	$C=m\sqrt{2g}$=coefficient for Francis' formula
1	0.248	0.3158	2.55
2	0.317	0.3217	2.58
3	0.390	0.3322	2.67
4	0.455	0.3410	2.73
5	0.521	0.3513	2.82
6	0.585	0.3607	2.89
7	0.653	0.3705	2.97
8	0.705	0.3740	3.00
9	0.766	0.3841	3.08
10	0.818	0.3946	3.16
11	0.882	0.4030	3.23
12	0.942	0.4119	3.30
13	0.999	0.4193	3.36
14	1.051	0.4231	3.39
15	1.103	0.4308	3.45
16	1.165	0.4353	3.49
17	1.209	0.4396	3.52
18	1.281	0.4447	3.57
19	1.330	0.4492	3.60
20	1.385	0.4527	3.63
21	1.446	0.4580	3.67

BAZIN'S

SERIES NO. 150

No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge	$C=m\sqrt{2g}$=coefficient for Francis' formula
1	0.248	0.3153	2.53
2	0.323	0.3297	2.65
3	0.379	0.3466	2.78
4	0.450	0.3515	2.82
5	0.512	0.3631	2.91
6	0.586	0.3692	2.96
7	0.637	0.3832	3.07
8	0.696	0.3892	3.12
9	0.751	0.3967	3.18
10	0.814	0.4063	3.26
11	0.869	0.4127	3.31
12	0.928	0.4203	3.37
13	0.982	0.4261	3.42
14	1.043	0.4328	3.47
15	1.095	0.4381	3.51
16	1.152	0.4439	3.56
17	1.215	0.4468	3.58
18	1.259	0.4529	3.63
19	1.315	0.4553	3.65
20	1.323	0.4555	3.65
21	1.380	0.4592	3.68
22	1.439	0.4645	3.73

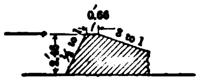
BAZIN'S

SERIES NO. 151

No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula
1	0.201	0.3373	2.71
2	0.240	0.3508	2.81
3	0.307	0.3476	2.79
4	0.391	0.3473	2.79
5	0.445	0.3641	2.92
6	0.514	0.3679	2.95
7	0.537	0.3720	2.98
8	0.573	0.3807	3.05
9	0.643	0.3850	3.09
10	0.695	0.3990	3.20
11	0.756	0.4047	3.24
12	0.800	0.4120	3.30
13	0.826	0.4131	3.31
14	0.867	0.4191	3.36
15	0.921	0.4226	3.39
16	0.975	0.4311	3.46
17	1.027	0.4373	3.50
18	1.090	0.4395	3.53
19	1.112	0.4450	3.57
20	1.140	0.4493	3.60
21	1.209	0.4499	3.61
22	1.248	0.4546	3.64
23	1.314	0.4587	3.68
24	1.353	0.4629	3.71
25	1.416	0.4669	3.75

BAZIN'S

SERIES NO. 153



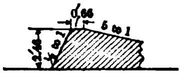
No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula
1	0.237	0.3406	2.73
2	0.301	0.3458	2.77
3	0.372	0.3476	2.79
4	0.373	0.3534	2.83
5	0.440	0.3613	2.90
6	0.505	0.3649	2.93
7	0.576	0.3747	3.00
8	0.637	0.3827	3.07
9	0.696	0.3866	3.10
10	0.701	0.3863	3.10
11	0.760	0.3930	3.15
12	0.768	0.3940	3.16
13	0.814	0.3996	3.20
14	0.879	0.4057	3.25
15	0.937	0.4107	3.29
16	0.983	0.4171	3.34
17	1.001	0.4159	3.33
18	1.056	0.4237	3.40
19	1.108	0.4253	3.41
20	1.170	0.4318	3.46
21	1.226	0.4341	3.48
22	1.290	0.4379	3.51
23	1.289	0.4392	3.52
24	1.347	0.4406	3.53
25	1.404	0.4463	3.58
26	1.436	0.4466	3.58

BAZIN'S

SERIES NO. 154

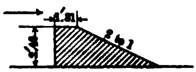
No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula
1	0.236	0.3370	2.70
2	0.306	0.3421	2.74
3	0.373	0.3521	2.83
4	0.447	0.3554	2.85
5	0.508	0.3677	2.95
6	0.577	0.3701	2.97
7	0.643	0.3787	3.04
8	0.706	0.3826	3.07
9	0.760	0.3950	3.17
10	0.823	0.3987	3.20
11	0.888	0.3996	3.20
12	0.946	0.4043	3.24
13	1.011	0.4095	3.28
14	1.075	0.4123	3.31
15	1.133	0.4186	3.36
16	1.195	0.4204	3.37
17	1.250	0.4245	3.40
18	1.310	0.4282	3.43
19	1.370	0.4295	3.45
20	1.430	0.4334	3.48

BAZIN'S SERIES NO. 156



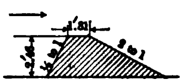
No. of experiment	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.245	0.3441	2.76
2	0.311	0.3488	2.79
3	0.382	0.3544	2.83
4	0.446	0.3611	2.90
5	0.508	0.3634	2.91
6	0.576	0.3675	2.95
7	0.638	0.3758	3.01
8	0.703	0.3821	3.06
9	0.764	0.3860	3.09
10	0.834	0.3897	3.12
11	0.888	0.3958	3.17
12	0.956	0.4035	3.24
13	1.018	0.4021	3.22
14	1.073	0.4119	3.30
15	1.133	0.4106	3.28
16	1.203	0.4126	3.31
17	1.269	0.4161	3.33
18	1.340	0.4194	3.36
19	1.394	0.4195	3.36
20	1,457	0.4227	3.38

BAZIN'S SERIES NO. 157



No. of experiment	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.241	0.3282	2.63
2	0.321	0.3238	2.60
3	0.396	0 3280	2.63
4	0.475	0.3299	2.65
5	0.547	0.3288	2.64
6	0.624	0.3317	2.66
7	0.696	0.3371	2.70
8	0.765	0.3406	2.73
9	0.826	0.3470	2.78
10	0.893	0.3496	2.81
11	0.957	0.3574	2.87
12	1.030	0.3591	2.88
13	1.093	0.3655	2.93
14	1.156	0.3698	2.97
15	1.216	0.3765	3.02
16	1.277	0.3801	3.05
17	1.337	0.3855	3.09
18	1.394	0.3883	3.11
19	1.456	0.3950	3.17
20	1.474	0.3989	3.20

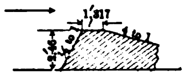
BAZIN'S SERIES NO. 158



No. of experiment	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.234	0.3479	2.79
2	0.312	0.3399	2.73
3	0.383	0.3463	2.77
4	0.457	0.3479	2.79
5	0.530	0.3499	2.81
6	0.600	0.3524	2.82
7	0.672	0.3559	2.86
8	0.733	0.3606	2.90
9	0.799	0.3633	2.91
10	0.860	0.3682	2.95
11	0.930	0.3744	3.00
12	0.984	0.3799	3.04
13	1.055	0.3866	3.10
14	1.125	0.3889	3.12
15	1.179	0.3923	3.14
16	1.243	0.3988	3.19
17	1.297	0.4025	3.22
18	1.361	0.4066	3.25
19	1.418	0.4112	3.30
20	1,457	0.4142	3.32

BAZIN'S

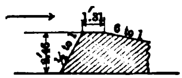
SERIES NO. 159



No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula
1	0.234	0.3333	2.68
2	0.304	0.3494	2.75
3	0.379	0.3515	2.82
4	0.387	0.3507	2.82
5	0.457	0.3498	2.81
6	0.516	0.3620	2.91
7	0.586	0.3544	2.84
8	0.599	0.3519	2.82
9	0.664	0.3571	2.87
10	0.670	0.3527	2.83
11	0.735	0.3591	2.88
12	0.797	0.3663	2.94
13	0.861	0.3727	2.99
14	0.876	0.3665	2.94
15	0.935	0.3654	2.93
16	0.994	0.3751	3.01
17	1.068	0.3777	3.03
18	1.126	0.3869	3.10
19	1.145	0.3798	3.05
20	1.198	0.3839	3.08
21	1.261	0.3878	3.11
22	1.320	0.3935	3.15
23	1.332	0.3906	3.13
24	1.389	0.3912	3.14
25	1.445	0.3978	3.19
26	1.456	0.3977	3.19

BAZIN'S

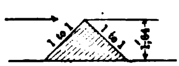
SERIES NO. 160



No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula
1	0.451	0.3505	2.81
2	0.522	0.3512	2.82
3	0.593	0.3541	2.84
4	0.663	0.3589	2.88
5	0.735	0.3598	2.89
6	0.798	0.3629	2.91
7	0.863	0.3641	2.92
8	0.930	0.3701	2.97
9	0.998	0.3731	2.94
10	1.074	0.3772	3.02
11	1.129	0.3784	3.03
12	1.193	0.3818	3.06
13	1.264	0.3842	3.08
14	1.326	0.3868	3.10
15	1.389	0.3919	3.14
16	1.457	0.3937	3.16

BAZIN'S

SERIES NO. 161



No. of experiment	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula
1	0.298	0.5373	4.31
2	0.354	0.5357	4.30
3	0.413	0.5308	4.26
4	0.472	0.5272	4.23
5	0.529	0.5260	4.22
6	0.581	0.5297	4.25
7	0.639	0.5289	4.24
8	0.693	0.5314	4.26
9	0.750	0.5331	4.28
10	0.808	0.5374	4.31
11	0.864	0.5373	4.31
12	0.919	0.5385	4.32
13	0.960	0.5405	4.34
14	0.992	0.5359	4.30
15	1.019	0.5374	4.31
16	1.056	0.5340	4.28
17	1.083	0.5323	4.27
18	1.118	0.5282	4.24
19	1.157	0.5196	4.17
20	1.187	0.5188	4.16
21	1.225	0.5139	4.12
22	1.263	0.5102	4.09
23	1.289	0.5118	4.11
24	1.326	0.5087	4.08
25	1.359	0.5086	4.08

BAZIN'S

SERIES NO. 163

No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.184	0.4746	3.81
2	0.244	0.4781	3.83
3	0.303	0.4786	3.84
4	0.366	0.4775	3.83
5	0.423	0.4779	3.83
6	0.486	0.4759	3.82
7	0.536	0.4816	3.86
8	0.593	0.4917	3.94
9	0.653	0.4880	3.91
10	0.702	0.4993	4.01
11	0.769	0.4966	3.98
12	0.827	0.5017	4.02
13	0.882	0.5006	4.02
14	0.949	0.5038	4.04
15	0.996	0.5062	4.06
16	1.056	0.5060	4.06
17	1.114	0.5052	4.05
18	1.171	0.5071	4.07
19	1.231	0.5076	4.07
20	1.285	0.5129	4.12
21	1.339	0.5199	4.17

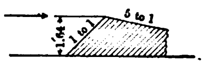
BAZIN'S

SERIES NO. 164

No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.244	0.4811	3.86
2	0.305	0.4861	3.90
3	0.367	0.4821	3.87
4	0.425	0.4867	3.90
5	0.482	0.4828	3.87
6	0.539	0.4826	3.87
7	0.592	0.4913	3.94
8	0.651	0.4915	3.94
9	0.702	0.4948	3.97
10	0.766	0.4990	4.00
11	0.817	0.5021	4.03
12	0.877	0.5046	4.05
13	0.939	0.5070	4.07
14	0.993	0.5111	4.10
15	1.052	0.5093	4.09
16	1.115	0.5136	4.12
17	1.162	0.5151	4.13
18	1.219	0.5172	4.15
19	1.277	0.5205	4.18
20	1.330	0.5217	4.19

BAZIN'S

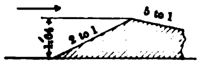
SERIES NO. 165



No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.337	0.4435	3.56
2	0.401	0.4447	3.56
3	0.464	0.4443	3.58
4	0.528	0.4433	3.55
5	0.593	0.4413	3.54
6	0.656	0.4422	3.54
7	0.720	0.4422	3.54
8	0.783	0.4433	3.55
9	0.843	0.4461	3.58
10	0.904	0.4506	3.61
11	0.969	0.4528	3.63
12	1.029	0.4531	3.63
13	1.090	0.4538	3.64
14	1.153	0.4553	3.65
15	1.217	0.4563	3.66
16	1.279	0.4584	3.68
17	1.341	0.4583	3.68
18	1.401	0.4601	3.69
19	1.448	0.4648	3.73

BAZIN'S

SERIES NO. 166



No. of experiment	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.330	0.4563	3.66
2	0.393	0.4548	3.65
3	0.455	0.4536	3.64
4	0.518	0.4538	3.64
5	0.575	0.4579	3.67
6	0.638	0.4579	3.67
7	0.699	0.4583	3.68
8	0.761	0.4596	3.69
9	0.821	0.4625	3.71
10	0.880	0.4663	3.74
11	0.947	0.4677	3.75
12	1.008	0.4681	3.76
13	1.066	0.4699	3.77
14	1.127	0.4714	3.78
15	1.187	0.4723	3.79
16	1.247	0.4731	3.80
17	1.306	0.4746	3.81
18	1.372	0.4748	3.81

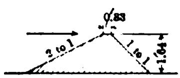
BAZIN'S

SERIES NO. 167

1	0.341	0.4087	3.28
2	0.398	0.4256	3.41
3	0.453	0.4367	3.50
4	0.509	0.4541	3.64
5	0.565	0.4577	3.67
6	0.616	0.4679	3.75
7	0.671	0.4753	3.81
8	0.725	0.4823	3.87
9	0.781	0.4917	3.94
10	0.828	0.4975	3.99
11	0.885	0.5046	4.05
12	0.930	0.5127	4.11
13	0.987	0.5176	4.15
14	1.044	0.5213	4.19
15	1.084	0.5293	4.25
16	1.142	0.5319	4.27
17	1.193	0.5349	4.29
18	1.245	0.5396	4.33
19	1.304	0.5431	4.36
20	1.354	0.5437	4.36

BAZIN'S

SERIES NO. 168

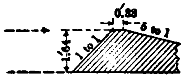


1	0.331	0.4137	3.32
2	0.392	0.4286	3 44
3	0.444	0.4435	3.56
4	0.503	0.4505	3.61
5	0.560	0.4592	8.68
6	0.619	0.4648	3.73
7	0.669	0.4733	3.80
8	0.732	0.4740	3.80
9	0.788	0.4801	3.85
10	0.839	0.4877	3.91
11	0.898	0.4915	3.94
12	0.954	0.4949	3.97
13	1.009	0.4999	4.01
14	1.059	0.5058	4.06
15	1.117	0.5080	4.08
16	1.168	0.5122	4.11
17	1.220	0.5189	4.16
18	1.275	0.5180	4.16
19	1.338	0.5205	4.18
20	1.386	0.5252	4.22

BAZIN'S
SERIES NO. 170

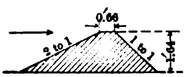
No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.351	0.4184	3.35
2	0.411	0.4300	3.45
3	0.468	0.4364	3.50
4	0.529	0.4488	3.60
5	0.585	0.4488	3.60
6	0.642	0.4576	3.67
7	0.701	0.4638	3.72
8	0.753	0.4689	3.76
9	0.811	0.4753	3.81
10	0.871	0.4796	3.85
11	0.926	0.4824	3.87
12	0.981	0.4896	3.93
13	1.009	0.4928	3.95
14	1.060	0.4948	3.97
15	1.115	0.4987	4.00
16	1.174	0.5027	4.03

BAZIN'S
SERIES NO. 172



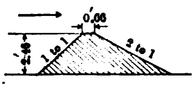
No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.356	0.3872	3.11
2	0.419	0.4025	3.23
3	0.488	0.4039	3.24
4	0.539	0.4143	3.32
5	0.602	0.4175	3.35
6	0.661	0.4223	3.39
7	0.723	0.4255	3.41
8	0.779	0.4333	3.48
9	0.838	0.4368	3.50
10	0.906	0.4414	3.54
11	0.962	0.4436	3.56
12	1.026	0.4460	3.58
13	1.086	0.4482	3.59
14	1.152	0.4499	3.61
15	1.204	0.4558	3.66
16	1.267	0.4586	3.69
17	1.325	0.4607	3.70
18	1.377	0.4670	3.75
19	1.449	0.4645	3.73

BAZIN'S
SERIES NO. 175



No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.360	0.3950	3.17
2	0.421	0.4061	3.25
3	0.487	0.4054	3.25
4	0.548	0.4119	3.30
5	0.608	0.4230	3.38
6	0.667	0.4281	3.43
7	0.726	0.4337	3.48
8	0.782	0.4396	3.52
9	0.837	0.4482	3.59
10	0.894	0.4554	3.65
11	0.949	0.4607	3.70
12	1.007	0.4649	3.73
13	1.061	0.4710	3.78
14	1.123	0.4747	3.81
15	1.174	0.4790	3.84
16	1.230	0.4811	3.86
17	1.287	0.4845	3.89
18	1.347	0.4869	3.91
19	1.387	0.4908	3.94

BAZIN'S
SERIES NO. 176

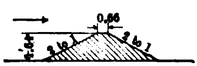


No. of experiment	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.237	0.3439	2.76
2	0.296	0.3418	2.74
3	0.365	0.3636	2.92
4	0.439	0.3684	2.95
5	0.494	0.3795	3.04
6	0.565	0.3867	3.10
7	0.618	0.3977	3.19
8	0.682	0.4021	3.22
9	0.733	0.4104	3.29
10	0.797	0.4175	3.35
11	0.861	0.4251	3.41
12	0.910	0.4296	3.45
13	0.974	0.4376	3.51
14	1.027	0.4409	3.53
15	1.083	0.4449	3.57
16	1.139	0.4518	3.62
17	1.196	0.4547	3.65
18	1.248	0.4586	3.68
19	1.303	0.4652	3.73
20	1.355	0.4676	3.75
21	1.420	0.4727	3.80

BAZIN'S
SERIES NO. 177

No. of experiment	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.367	0.3863	3.10
2	0.433	0.3872	3.18
3	0.497	0.3993	3.20
4	0.556	0.4050	3.25
5	0.617	0.4127	3.31
6	0.680	0.4180	3.35
7	0.739	0.4242	3.40
8	0.794	0.4309	3.46
9	0.850	0.4389	3.52
10	0.906	0.4482	3.59
11	0.965	0.4562	3.66
12	1.019	0.4621	3.71
13	1.077	0.4658	3.74
14	1.129	0.4716	3.79
15	1.184	0.4753	3.81
16	1.243	0.4779	3.83
17	1.296	0.4840	3.88
18	1.346	0.4867	3.91
19	1.386	0.4924	3.95

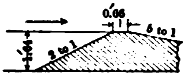
BAZIN'S
SERIES NO. 179



No. of experiment	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.364	0.3930	3.15
2	0.431	0.3997	3.20
3	0.490	0.4054	3.25
4	0.558	0.4091	3.28
5	0.614	0.4180	3.35
6	0.674	0.4244	3.40
7	0.732	0.4277	3.43
8	0.790	0.4331	3.47
9	0.851	0.4399	3.53
10	0.908	0.4480	3.59
11	0.968	0.4536	3.64
12	1.023	0.4588	3.68
13	1.078	0.4641	3.72
14	1.134	0.4656	3.74
15	1.190	0.4703	3.77
16	1.247	0.4740	3.80
17	1.298	0.4796	3.85
18	1.357	0.4840	3.88
19	1.403	0.4884	3.92

BAZIN'S

SERIES NO. 181



No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis'. formula.
1	0.219	0.3789	3.04
2	0.290	0.3858	3.09
3	0.356	0.3863	3.10
4	0.423	0.3900	3.13
5	0.486	0.3993	3.20
6	0.554	0.3970	3.18
7	0.621	0.4030	3.23
8	0.677	0.4092	3.28
9	0.743	0.4105	3.29
10	0.797	0.4153	3.33
11	0.861	0.4210	3.38
12	0.920	0.4267	3.42
13	0.983	0.4311	3.46
14	1.044	0.4345	3.49
15	1.111	0.4394	3.52
16	1.159	0.4413	3.54
17	1.228	0.4447	3.57
18	1.278	0.4484	3.59
19	1.342	0.4518	3.62
20	1.403	0.4534	3.63
21	1.463	0.4578	3.67

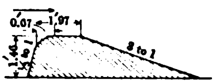
BAZIN'S

SERIES NO. 182

No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis'. formula.
1.	0.280	0.3927	3.15
2	0.403	0.4012	3.22
3	0.526	0.4150	3.33
4	0.641	0.4309	3.46
5	0.761	0.4448	3.56
6	0.867	0.4584	3.68
7	0.989	0.4654	3.73
8	1.091	0.4749	3.81
9	1.210	0.4821	3.87
10	1.320	0.4870	3.91
11	1.413	0.4966	3.99

BAZIN'S

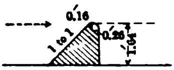
SERIES NO. 188



No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis'. formula.
1	0.289	0.3758	3.01
2	0.425	0.3835	3.08
3	0.563	0.3836	3.08
4	0.697	0.3925	3.15
5	0.825	0.3962	3.19
6	0.953	0.4045	3.24
7	1.080	0.4120	3.30
8	1.201	0.4193	3.36
9	1.322	0.4274	3.43
10	1.431	0.4388	3.52
11	1.546	0.4432	3.55

BAZIN'S

SERIES NO. 189



No. of experiment.	h=depth on crest, in feet.	m=coefficient of discharge	$C=m\sqrt{2g}$= coefficient for Francis' formula.	
1	0.248	0.4605	3.69	Adhering Nappe.
2	0.300	0.4834	3.83	
3	0.356	0.4967	3.99	
4	0.406	0.5090	4.08	
5	0.457	0.5198	4.17	
6	0.511	0.5331	4.28	
7	0.557	0.5389	4.32	
8	0.607	0.5517	4.43	
9	0 655	0.5563	4.46	
10	0.706	0.5635	4.52	
11	0.706	0.5624	4.51	Nappe wetted underneath.
12	0.756	0.5664	4.54	
13	0.811	0.5706	4.58	
14	0.864	0.5698	4.57	
15	0.923	0.5677	4.55	
16	0.979	0.5681	4.56	
17	1.042	0.5644	4.53	
18	1.097	0.5612	4.50	
19	1.151	0.5629	4.52	
20	1.227	0.5550	4.45	
21	1.274	0.5572	4.47	

BAZIN'S

SERIES NO. 193

No. of experiment.	h=depth on crest, in feet.	m=coefficient of discharge	$C=m\sqrt{2g}$= coefficient for Francis' formula.
1	0.260	0.4354	3.49
2	0.317	0.4496	3.61
3	0.374	0.4635	3.72
4	0.429	0.4716	3.79
5	0.476	0.4891	3.92
6	0.534	0.4924	3.95
7	0.585	0.5049	4.05
8	0.634	0.5106	4.10
9	0.684	0.5238	4.20
10	0.736	0.5293	4.25
11	0.780	0.5418	4.35
12	0.836	0.5480	4.40
13	0.886	0.5559	4.46
14	0.932	0.5628	4.52
15	0.979	0.5644	4.53
16	1.035	0.5710	4.58
17	1.093	0.5675	4.55
18	1.148	0.5663	4.54
19	1.201	0.5695	4.57
20	1.266	0.5665	4.54

General Résumé of Bazin's Experiments.

This chapter has also been translated from Bazin's final paper in *Annales des Ponts et Chaussées* for 1898.*

Classification of the Different Species of Nappes.—When water is discharged through a submerged orifice, it is well known that the discharge Q varies according to the nature of the orifice. The coefficient m, in the formula $Q = m\, c \sqrt{2\, g\, h}$, must be determined for each particular case. This coefficient varies, however, only within relatively narrow limits.

The conditions in the case of a weir are much more complex. The influenee of the shape (width of crest, degree of inclination of the up-stream and down-stream faces of the weir, etc.) always enters, but other factors conspire to make the value of the coefficient m, in the formula $Q = m\, l\, h \sqrt{2\, g\, h}$, vary within much wider limits than for discharge through an orifice.

The sloping nappe of a weir differs from the vein issuing from an orifice in that it may assume a variety of perfectly distinct forms. These forms constitute, in reality, as many distinct cases, each of which it is necessary to study separately, since, by confusing them one must necessarily expose himself to serious errors. It is proper, at the start to classify these different forms and to make known their distinctive characteristics.

Free Nappes.—The most simple and definite case is that of a sharp-crested weir without lateral contractions, in which the nappe falls

* Bazin's formulas have been changed, so as to make English measures applical ble, by the introduction of a conversion factor when necessary. The mathematica symbols are as follows:

h = head on crest of weir, in feet;
u = mean velocity in channel above the weir, in feet;
a = a constant (replaces α used by Bazin);
K = a constant;
g = acceleration of gravity, = 32.2;
l = length of crest of weir, in feet;
n = a constant (replaces u used by Bazin);
p = height of crest of weir above bottom of channel, in feet;
m = coefficient of discharge over a given weir;
m' = corresponding coefficient for a standard weir;
Q = discharge, in cubic feet per second;
h_e = difference of elevation of crest of weir and of water on the down-stream side, in feet;
c = width of crest of a flat-crested weir, in feet;
R = radius of curvature of a weir rounded at the back, in feet;
$z = h - h_e$, in feet.

freely in the air, its lower surface always subject to the pressure of the atmosphere. The lateral contraction may be suppressed by making the length of the weir equal to the width of the canal leading to it, which is supposed to have vertical walls. Immediately below the crest, these walls should be constructed in such a manner as to permit free access of air beneath the nappe, which then represents a portion of a nappe of indefinite length. The resulting phenomena of flow are perfectly constant; the nappe, independent of any influence of the water below the weir, permits of a precise determination of the coefficient m.

Depressed Nappes and Nappes Wetted Underneath.—When the walls of the canal in which the weir is placed are not constructed in such a manner as to maintain free access of air beneath the nappe, the phenomena of discharge become more complicated, and the form of the nappe, remarkably constant in the case where it falls freely over the weir, undergoes considerable modification, according to the amount of discharge.

When the head does not exceed a certain limit, the nappe remains detached from the face of the weir, imprisoning underneath it a volume of air at a pressure inferior to that of the external atmosphere. At the same time the water in the space between the foot of the nappe and that of the weir rises to a level above that of the stream below the weir. We have, then, a species of free nappe, modified and depressed by the excess of external pressure. The discharge over such a sharp-crested weir is, for equal heads, slightly greater than over a true free nappe. The difference increases as the volume of the imprisoned air diminishes.

Depressed nappes are not very stable; the accidental entrance of air from time to time produces variations in both the interior pressure and the discharge. When the air has entirely disappeared, the nappe takes a more definite form, which may be designated as wetted underneath. The overflowing nappe encloses a small portion of turbulent water, which does not partake of the translatory movement of the vein, properly speaking.

The occurrence of a nappe wetted underneath may be independent of the level of the water on the down-stream side of the weir; or, on the contrary, it may be influenced by this level, every modification of which then reacts on the discharge over the weir.

The former is the case when the overfall is followed by a rapid which terminates in an abrupt ressault, below which the flow takes place in accordance with the condition of the channel. The position of the ressault is without influence on the discharge, provided it does not enclose the foot of the nappe.

In the second case, that is to say, when the foot of the nappe is more or less enclosed in the water in the channel below the weir, it is not easy to separate the influence of the water in the channel, for the discharge may be modified, although this water does not rise to the level of the crest of the weir.

Undulating Nappes.—When the level of the water in the channel below the weir is raised to the height of the weir crest, the wetted nappe retains its form as long as the difference in level, or fall of the water from the up-stream to the down-stream side of the weir, does not descend below a certain limit. Its characteristic profile persists, although in part concealed by its immersion in the channel on the down-stream side, but, where the fall or difference in level is progressively diminished, a moment comes when the nappe returns suddenly to the surface, undulating in the meanwhile. This change, although very apparent, does not exert any important influence on the value of the coefficient of discharge.

Adhering Nappes.—The forms of nappes thus far considered are those most ordinarily encountered. Another form exists, which appears under certain conditions, in which the nappe, instead of enfolding a small mass of turbulent water having no translatory movement, as in the case of the nappe wetted underneath, is, on the contrary, in close contact with the face of the weir. It presents then, in certain cases, some interesting particulars, and to this remarkable form there often corresponds a considerable increase of the coefficient of discharge.

The *ensemble* of the phenomena of discharge is very complex, and one cannot often determine the discharge of a weir with precision without previously knowing under which particular form of nappe the discharge took place. Taking, for example, a sharp-crested weir 0.75 m. (2.46 ft.) high, we have shown that for the same head of 0.20 m. (0.656 ft.) the nappe may assume four very distinct forms, to which correspond the following different values of the coefficient m in the formula $Q = m\,l\,h\,\sqrt{2\,g\,h}$:

	m.	$m\sqrt{2g}$.
(1) Free nappe, under surface always subjected to atmospheric pressure..................	0.433	3.47
(2) Depressed nappe, imprisoning a certain volume of air at a pressure below that of the atmosphere..............................	0.460	3.69
(3) Nappe wetted underneath, no air imprisoned, level of water on down-stream side 0.125 m. (0.42 ft.) below crest of weir..............	0.497	3.99
(4) Adhering nappe, the ressault being at a distance from the foot of the nappe, which is completely exposed......................	0.554	4.45

Sharp-Crested Weirs.

Free Nappes.—When the nappe, in flowing over a sharp-crested weir, has its lower surface in free communication with the atmosphere, the only element which modifies the coefficient m is the mean velocity u in the channel leading to the weir. In order to take this into consideration in the formula, h is replaced by $h + a\frac{u^2}{2g}$. The formula becomes then, representing by n the modified coefficient m,

$$Q = n\,l\left(h + a\frac{u^2}{2g}\right)\sqrt{2g\left(h + a\frac{u^2}{2g}\right)}$$

$$= n\,l\,h\sqrt{2gh}\left(l + a\frac{u^2}{2gh}\right)^{\frac{3}{2}}$$

or, approximately, considering that $\frac{u^2}{2gh}$ is a fraction rarely exceeding a few centimeters,

$$Q = n\,l\,h\sqrt{2gh}\left(l + \frac{3}{2}\,a\frac{u^2}{2gh}\right)$$

This expression is not in a form convenient for use, since the velocity u depends on the discharge to be determined. If p be used to designate the height of the weir above the bottom of the channel, the wetted section of the canal is $l\,(h + p)$, and we have

$$\frac{u^2}{2g} = \frac{Q^2}{2gl^2(h+p)^2}\left(\frac{l}{(3.28\,l)^2}\right)$$

or, simply replacing Q by its value $m\,l\,h\sqrt{2gh}$, we have,

$$\frac{u^2}{2gh} = m^2\left(\frac{h}{h+P}\right)^2 \qquad (1)$$

Letting, for short $\left(\frac{3}{2}\, a\, m^2\right) = K$, the above expression takes the more practical form

$$Q = \frac{u}{3.28\, l}\left[l + K\left(\frac{h}{h+P}\right)^2\right] l\, h \sqrt{2gh} \dots\dots\dots \quad (2)$$

We have determined the coefficients a, K and n by comparative experiments on five weirs of different heights. a and K are not perfectly constant, but one may take, as a mean, $a = \frac{5}{3}$; $K = 0.55$. As to n, its value, which corresponds to the limiting case of no velocity of approach, cannot be measured directly, since it is impossible to completely suppress this velocity. Its influence, however, becomes insignificent in a high weir. The coefficient n decreases slowly as the head increases, as shown below:

Head, in feet	= 0.164, 0.328, 0.656, 0.984, 1.312, 1.640.
Corresponding values of n	= 0.448, 0.432, 0.421, 0.417, 0.414, 0.412.

When h is over 0.328 ft. its value is represented with sufficient precision by the formula,

$$n = 0.405 + \left(\frac{0.003}{h}\right) 3.28\, l \dots\dots\dots \quad (3)$$

Adopting for K the value 0.55, Formula (2) becomes

$$m = n\left[l + 0.55\left(\frac{h}{h+P}\right)^2\right] \dots\dots\dots \quad (4)$$

In most cases, where the head falls between 0.10 m. (0.328 ft.) and 0.30 m. (0.984 ft.), n may be taken at the constant value 0.425, and taking $K = \frac{1}{2}$ simply, the expression for m becomes

$$\left.\begin{aligned} m &= 0.425\left[l + \frac{1}{2}\left(\frac{h}{h+P}\right)^2\right] \\ &= 0.425 + 0.212\left(\frac{h}{h+P}\right)^2 \end{aligned}\right\} \dots\dots\dots \quad (5)$$

which will answer for all practical cases where errors of 2 to 3% are permissible. It is this coefficient of discharge m, perfectly determined by the head h and the height p of the weir, which has been used for comparison. Instead of considering on the other weirs the absolute values of the coefficient m, we have compared them with the coefficient m' for a free nappe, for the same head on a sharp-crested weir of the same height.

This substitution of the ratio $\frac{m}{m}$, for the absolute values of m eliminates, in a large measure, at least, the influence of velocity of approach and facilitates greatly the discussion of results.

In what precedes, the up-stream face of the weir has been supposed to be a vertical plane. If it is inclined, the values of the coefficient m will be modified. The coefficient is diminished when the plane of the dam is inclined up stream, but, if, on the contrary, the plane of the dam is inclined down stream, the coefficient increases to a maximum which corresponds nearly to an inclination of 30° (equals a batter of $1\frac{3}{4}:1$ on the back). The ratio between the coefficients corresponding to two different inclinations is sensibly constant for all heads, so that one may obtain the coefficient m for a sharp-crested weir at any inclination by multiplying by a constant, or modulus, the corresponding coefficient for a vertical weir, as is indicated in Table No. 3.

This ratio increases regularly from an inclination of 45° toward the up-stream side, to approximately 30° toward the down-stream side where the maximum occurs, from which point the discharge does not take place in the normal manner, since the liquid vein in its passage over the crest, instead of being freely contracted, is guided by the incline of the weir on which it rests in immediate contact.

TABLE No. 3.

		Modulus by which to multiply the coefficient for a vertical weir.
Up-stream inclination of the weir.	1 horizontal to 1 vertical	0.93
	2 " " 3 "	0.94
	1 " " 3 "	0.96
Vertical weir		1.00
Down-stream inclination of the weir.	1 horizontal to 3 vertical	1.04
	2 " " 3 "	1.07
	1 " " 1 "	1.10
	2 " " 1 "	1.12
	4 " " 1 "	1.09

Sharp-Crested Weirs. Nappes Depressed and Wetted Underneath.—When the air is not admitted freely underneath the nappe, the phenomena become more complicated. The nappe may be either depressed, as a result of air being imprisoned underneath at less than atmospheric pressure, or it may be wetted underneath without

containing any air. The discharge for a depressed nappe is slightly in excess of that for a free nappe. The difference may rise to nearly 10% at the moment when the nappe, the confined air being nearly exhausted, is at the point of assuming the form of a nappe wetted underneath. The accidental entrance of air from time to time may vary the discharge a little. The wetted nappes are more uniform. It is important to distinguish two cases according as the ressault, which is produced below the nappe, is at a distance from its foot, or partly encloses it.

First Case. Ressault at a Distance.—The coefficient m may be deduced from the coefficient m' for a free nappe by the relation

$$m = m' \left(0.878 + 0.128 \frac{P}{h}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The ratio $\frac{p}{h}$ can only have certain values, as experience has shown that it does not exceed 2.5, because the form of nappe wetted underneath does not continue if the head is less than 0.4 p. For the maximum value $\frac{p}{h} = 2.5$, we have, very nearly, $m = 1.20\ m'$; and when $h = p$, m becomes sensibly equal to m'. Finally, m is a little greater than m' when h surpasses p.

If the above formula be applied to weirs of different heights, it may be shown that for the same value of $\frac{p}{h}$ the absolute values of m do not differ greatly from those given by the equation

$$m = 0.470 + 0.0075 \frac{P^2}{h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

which permits us to find an absolute value of the coefficient m without using the ratio $\frac{m}{m'}$.

Second Case. The Ressault Enclosing Part of the Nappe.—It is necessary to take into account the level of the water below the weir, and, if we designate by h_l the difference of level of the crest of the weir and of the water below, the value of m becomes

$$m = m' \left[1.06 + 0.16 \left(\frac{h_l}{P} - 0.05\right) \frac{P}{h} \right] \quad \ldots\ldots\ldots\ldots (8)$$

In this formula, h_l is to be taken as minus when the level of the water on the down-stream side is below crest, and as plus when it is above the crest. The formula can only be applied within certain limits of

h_l. If the difference in level of water above and below the weir be increased, a moment comes when the ressault is driven back from the foot of the nappe until it ceases to enclose it and changes then to the preceding case. This pushing back of the ressault takes place when the total fall, $(h + h_l)$ is approximately equal to $\frac{3}{4}p$. That is to say, for a given head h the greatest admissible value of h_l is $\left(\frac{3}{4}h - p\right)$. On the other hand, when the head h is insufficient to throw back the ressault, it is necessary that the level below the weir be sufficient to sustain the foot of the nappe and to prevent the introduction of air, which would cause the nappe to return to the depressed form. The preceding formula may be simplified by suppressing the small term, 0.55, in the parentheses, and, for compensation, slightly diminishing the two other coefficients. It then becomes

$$m = m' \left(1.05 + 0.15 \frac{h_l}{h} \right) \dots\dots\dots\dots\dots\dots (9)$$

Sharp-Crested Weirs. Adhering Nappes.—The nappe may also take, though very rarely, a particular form, the production of which depends on the width of the dam and the form of the upper part supporting the sharp crest. The nappe becomes completely attached to the downstream face of the dam without the interposition of air. The coefficient of discharge then becomes very large and may rise as high as 1.30 m', which corresponds to an absolute value of the coefficient $m = 0.55$ or 0.56. Adhering nappes present curious particulars, but as they only occur exceptionally in practice, we may simply refer to the special studies made of them, included in *Annales des Ponts et Chaussées* for 1891.

Beam Weirs. Free Nappes.—Beam weirs are constructed of square timbers of the same cross-sectional dimensions, placed one upon another to the desired height. The back and front faces of the weir are vertical planes, but the crest, instead of being reduced to a sharp edge, presents a horizontal surface, the width of which equals the thickness of the timbers. This circumstance completely modifies the conditions of discharge, and, while this form of weir is readily constructed, it may, unfortunately, give considerable error in the gaugings.

The free nappes appear under two distinct forms, according as the

nappe is in contact with the horizontal crest, or becomes detached at the back edge in such a manner as to flow over the crest without touching the down-stream edge. In the second case the influence of the flat crest evidently disappears and the discharge is like that over a sharp-crested weir. The nappe assumes this form when the head exceeds twice the width of c of the crest, measured in the direction of discharge, but it may occur whenever the head exceeds $\frac{3}{2}c$. Between these limits the nappe is in a state of instability; it tends to detach itself from the crest and may do so under the influence of any external disturbance, as, for example, the entrance of air or the passage of a floating body over the weir.

When the nappe adheres to the crest, the coefficient m depends chiefly on the ratio $\frac{h}{c}$ and may be represented by the formula,

$$m = m' \left(0.70 + 0.185 \frac{h}{c}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

m varies, as a consequence, very rapidly. We have,

When $\frac{h}{c} = 0.50$	$\frac{m}{m'} = 0.79$	
" $= 1.00$	" $= 0.88$	
" $= 1.50$	" $= 0.98$	or 1.0 if the nappe is
" $= 2.00$	" $= 1.07$	detached.

When $\frac{h}{c}$ exceeds 2.00, $\frac{m}{m'} = 1.00$. It will be seen that between $h = \frac{3}{2c}$ and $h = 2c$, $\frac{m}{m'}$ may vary from 0.98 to 1.07, or nearly a tenth in value, or, it may remain constantly equal to unity, according as the nappe is attached to or free from the crest.

Very Wide Crests.—When the width of the crest is considerable, 1 or 2 m., for example, the foregoing formula is still applicable, giving results within a few per cent. The value of $\frac{h}{c}$ then reduces to a few tenths and the ratio $\frac{m}{m'}$ also becomes much smaller, so that m may not exceed 0.35. For example, at a head of 0.45 m. (1.476 ft.) on a weir with a flat crest 2 m. wide, $\frac{m}{m'} = 0.755$, which corresponds to an absolute value of $m = 0.337$. The formula gives $\frac{m}{m'} = 0.732$ and as a result, $m = 0.326$.

Effect of Rounding the Crest at the Back.—A slight rounding of the back edge of the crest very sensibly modifies the discharge. Fteley and Stearns have shown that rounding the back edge of the crest to a radius R augments the discharge by an amount equal to that given by a head increased by 0.7 R. This is equivalent to increasing the coefficient m in the ratio of $h^{\frac{3}{2}}$ to $(h + 0.7\ R)^{\frac{3}{2}}$ or nearly in the ratio of 1 to 1 $+ \frac{R}{h}$. The radius R in their experiments did not exceed 0.039 ft., and it is clear that this approximate mode of correction will not apply to cases where the radius is notably greater. We have experimented on two weirs, respectively 0.80 m. (2.624 ft.) and 2.00 m. (6.56 ft.) in width, with crests rounded at the back to a radius of 0.10 m. (0.328 ft.), and this modification has had the effect of increasing the discharge 14% on the first of these weirs and 12% on the second. A simple rounding of 1 or 2 cm. radius, such as results from wear on timbers of ordinary dimensions, is by no means negligible from the point of view of the resulting discharge.

A weir with a crest 2 m. wide, and rounded at the back, gave, for the greatest head used in the experiments, $m = 0.373$, a value differing little from that indicated by theory for the case of a nappe flowing in filaments parallel to the horizontal surface of the crest. This hypothesis may not, however, be realized experimentally in more than a very imperfect manner, as the surface of the nappe undulates continually.

Beam Weirs. Nappes Depressed and Wetted Underneath.—The depressed nappes do not differ greatly from the free. The coefficient is at first less than for a free nappe, but approaches it progressively in value and finally exceeds it slightly. It differs in this respect from a sharp-crested weir, for which the coefficient for a depressed nappe is always superior to that for a free nappe. It makes no difference, as to this, whether the nappe clings to the flat crest or is detached from it. In either case the coefficient differs little from that for a sharp-crested weir. The effect of adherence to the flat crest appears again for a nappe wetted underneath, with this added difficulty, that the moment of detachment underneath the water is not apparent, and does not correspond to any constant value of $\frac{h}{c}$. In other words, it may take place either preceding or following the formation of the

nappe wetted underneath. It is necessary, in this regard, to distinguish two cases according as the height p of the dam is greater or less than about five times the width c of the crest. When p is greater than $5\,c$, the nappe detaches itself from the flat crest before it becomes wetted underneath, and in the intermediate state does not differ greatly from that for a sharp-crested weir. When, on the other hand, p is less than $5\,c$, the nappe does not detach itself from the flat crest before becoming wetted underneath, but is very unstable at the moment of this transformation.

So long as the nappe adheres to the crest, this influence predominates, and Formula (10) is most nearly applicable to the nappe wetted underneath. On the other hand, when the nappe is detached from the flat crest, the conditions of discharge approach more nearly those for a sharp crest, to which Formula (6) may be applied. The two formulas give the same value when the head exceeds a certain limit

$$h_l = \frac{c}{2}\left(1 + \sqrt{\frac{3\,p}{c}}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

For heads less than h_l, Formula (10) gives values of m slightly too small, never differing from those of the experiments, however, by more than 3 or 4 per cent. When the head exceeds h_l, one must take recourse to the other formula, although it, likewise, gives values which are too small. The difference, rather more important in this case, attains 8% as a maximum, after which it diminishes rapidly for increased heads. This maximum corresponds to the moment when the nappe is at the point of detaching itself from the flat crest. After it has become detached, the influence of width of crest disappears and Formula (6) applies with a very close degree of approximation.

It will be seen that the flat crest has the effect of doubling each species of nappe, in that two formulas must be applied according as the nappe clings to or is detached from the flat surface.

Weirs with Wide Crests, and Batter on the Faces.—The phenomena of discharge become much more complex for weirs, such as are often found in practice, with batters of greater or less inclination on the front and back faces. The influence of the flat crest, which exerts itself in a weir built up of square timbers, is joined to that of the slope of the faces. The inclination of the up-stream face, by reduc-

ing the contraction, has the effect of increasing the discharge. While that of the down-stream face has, ordinarily, the same effect as increasing the width of the flat crest, that is to say, it diminishes the discharge. The coefficient m, then, in each particular case, depends not only on the head, but on the width of the crest and the degree of inclination of the faces. It is, therefore, exceedingly variable, and each type demands a special study.

Rounding the back edge of the crest reduces the contraction considerably and may increase the value of m 10 or 15 per cent. Considering, finally, the class of weirs with completely curved profiles, such as are occasionally encountered in hydraulic practice, the value of m may attain a relatively high figure. The coefficients for such cases have not been arranged in comparative tables, but enough particular cases are given to serve as a guide in practice. It is clearly impossible to establish a general formula which will take account of all the elements that enter to affect the discharge.

Drowned Weirs.—We have given, in discussing the experiments on sharp-crested weirs drowned by the water on the down-stream side, two formulas; one of which applies to cases where the weir is not deeply drowned. The other, which is more general in its application, is

$$m = m^1 \left(1.08 + 0.18 \frac{h_l}{p}\right) \sqrt[3]{\frac{z}{h}} \dots\dots\dots\dots\dots (11)$$

The two formulas mentioned have been so established as to represent in the best possible manner the experiments from which they have been deduced. In cases where a less precise approximation will suffice, Formula (11) may be made applicable, by altering slightly its coefficients, as follows:

$$m = 1.05\, m' \left[1 + \frac{1}{5} \left(\frac{h_l}{p}\right)\right] \sqrt[3]{\frac{z}{h}} \dots\dots\dots\dots\dots (12)$$

This new expression is practically equivalent to the two others and gives the same values within 1 or 2%, except when the ratios $\frac{h}{p}$ and $\frac{h_l}{p}$ are very small. The difference may then be as much as 4 or 5%, but in this case the determination of the coefficient m is always very uncertain.

The effect of drowning is not the same for a wide-crested weir.

Raising the plane of the water below the weir, which in the case of a sharp-crested weir affected the flow on the up-stream side before the water below had reached the height of the crest, does not commence to take effect on a wide-crested weir until after the level of the water on the down-stream side is considerably above the crest; and the greater the width of the crest, the less is its ultimate effect. In our experiments on a crest 2 m. in width we have shown that the water on the down-stream side must rise to a height above the crest equal to $\frac{5}{6}$ of the head h before it affects the level on the up-stream side.

When the plane surface, which forms the crest of a weir, is very wide, it constitutes a sort of channel, and, in a measure, as the length of this channel is increased, the conditions of discharge depart from those which pertain to a weir, properly speaking, and approach those for a channel with a horizontal bottom.

The Cornell University Experiments.

At the beginning of the study of Bazin's work it was the writer's opinion that his coefficients could be fairly extended to depths on the crest of about 4 ft. without material error, and on this basis a number of discharge curves were worked out in the manner to be described. On further study, however, it seemed probable that some of Bazin's Series, especially Nos. 130 and 135 and a few others might be somewhat too high for deep flows, for the reason that at depths on the crest from 0 up to about 0.6 to 1.0 ft. the nappes were depressed and adherent, and above 0.6 to 1.0 ft. were wetted underneath, thus indicating that probably the conditions of the experiments were such as not to insure the free admission of air beneath the nappes, this condition leading to higher flows than with air freely admitted. Or, on the other hand, as Bazin himself points out, the limit of perfect detachment may not have been reached in his experiments.

Again, Bazin's weirs were constructed with closed fronts, thus offering an opportunity for adhering nappes, while in actual practice, for sections corresponding to Series Nos. 130 and 135, the water generally flows over a lip, the nappe dropping into a free air space below. This general condition is illustrated by the dams shown on several of the illustrative figures following. The conditions at the ends of such dams are such as to usually permit the free admission of air.

CORNELL UNIVERSITY
EXPERIMENT NO. 1
BAZIN'S SERIES NO. 130
MAY 20TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 7.

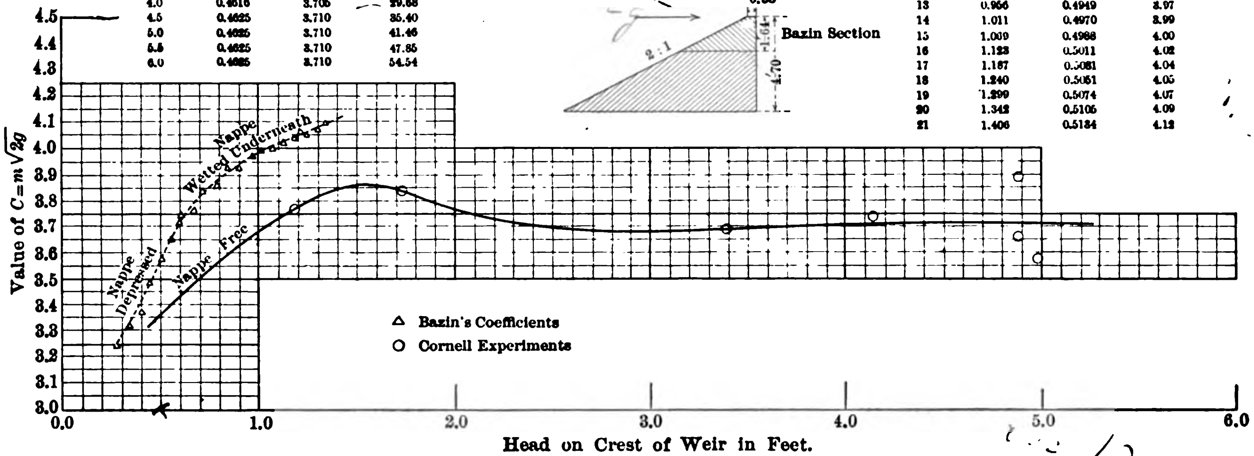
Limiting heads, 1.19 ft. and 4.97 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per foot.
0.5	0.4180	3.355	1.20
1.0	[illegible]	[illegible]	[illegible]
1.5	0.4473	[illegible]	6.59
2.0	0.4686	3.770	10.66
2.5	0.4598	3.690	14.59
3.0	0.4585	3.680	19.13
3.5	0.4598	3.690	24.25
4.0	0.4616	3.706	29.68
4.5	0.4625	3.710	35.40
5.0	0.4625	3.710	41.46
5.5	0.4625	3.710	47.85
6.0	0.4625	3.710	54.54

Bazin Section

BAZIN'S
SERIES NO. 130

No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C=m\sqrt{2g}$ = coefficient for Francis' formula.	
1	0.274	0.4087	3.24	Nappe depressed or adhering, but unstable.
2	0.336	0.4138	3.32	
3	0.396	0.4218	3.37	
4	0.452	0.4339	3.48	
5	0.510	0.4468	3.57	
6	0.565	0.4546	3.65	
7	0.618	0.4663	3.74	Nappe wetted underneath and attached to flat crest.
8	0.672	0.4694	3.76	
9	0.728	0.4769	3.83	
10	0.785	0.4801	3.85	
11	0.837	0.4882	3.92	
12	0.898	0.4893	3.92	
13	0.966	0.4949	3.97	
14	1.011	0.4970	3.99	
15	1.069	0.4988	4.00	
16	1.123	0.5011	4.02	
17	1.187	0.5031	4.04	
18	1.240	0.5051	4.05	
19	1.290	0.5074	4.07	
20	1.342	0.5106	4.09	
21	1.406	0.5134	4.12	



CORNELL UNIVERSITY

EXPERIMENT NO. 2.

BAZIN'S SERIES NO. 135.

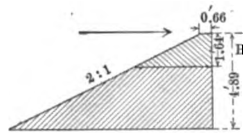
MAY 24TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 7.

Limiting heads, 0.341 ft. and 5.06 ft.

Head, in feet.	m	$C = m\sqrt{2g}$	Q per foot.
0.5	0.4012	3.220	1.15
1.0	0.4285	3.440	3.44
1.5	0.4474	3.590	6.59
2.0	0.4564	3.663	10.85
2.5	0.4598	3.690	14.59
3.0	0.4585	3.680	19.12
3.5	0.4598	3.690	24.15
4.0	0.4615	3.705	29.65
4.5	0.4625	3.710	35.40
5.0	0.4625	3.710	41.46
5.5	0.4625	3.710	47.84
6.0	0.4625	3.710	54.54

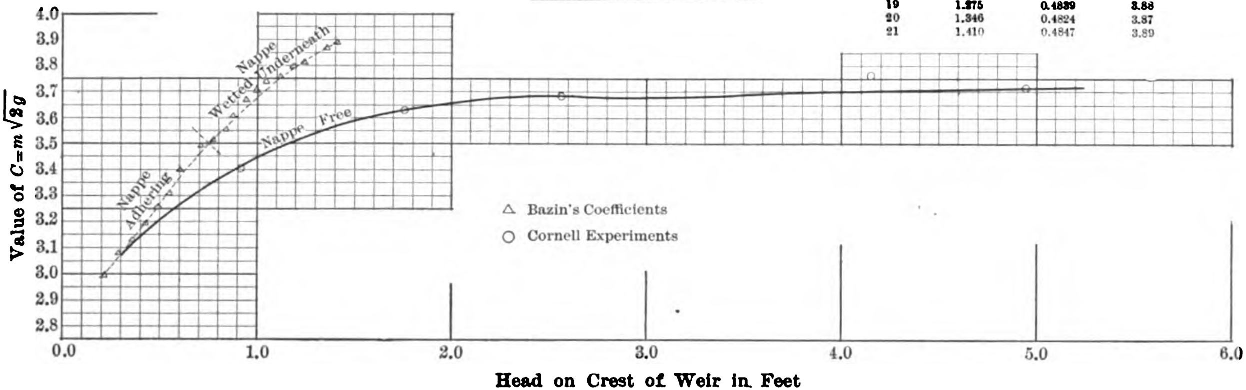


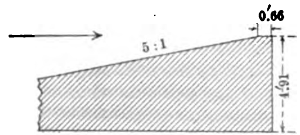
Bazin Section

BAZIN'S

SERIES NO. 135.

No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.	
1	0.224	0.3727	2.99	Nappe depressed.
2	0.292	0.3839	3.08	
3	0.353	0.3912	3.13	
4	0.422	0.3961	3.19	
5	0.480	0.4071	3.26	
6	0.541	0.4127	3.31	
7	0.597	0.4243	3.40	
8	0.657	0.4264	3.42	Nappe wetted
9	0.712	0.4347	3.49	underneath.
10	0.773	0.4394	3.52	
11	0.829	0.4439	3.55	
12	0.892	0.4496	3.61	
13	0.939	0.4575	3.67	
14	1.011	0.4614	3.70	
15	1.063	0.4657	3.74	
16	1.119	0.4696	3.76	
17	1.175	0.4740	3.80	
18	1.230	0.4757	3.82	
19	1.275	0.4839	3.88	
20	1.346	0.4824	3.87	
21	1.410	0.4847	3.89	





CORNELL UNIVERSITY

EXPERIMENT NO. 3.

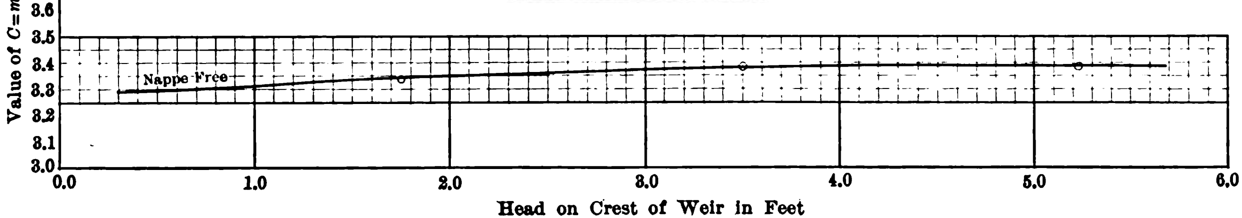
MAY 26TH, 1899.

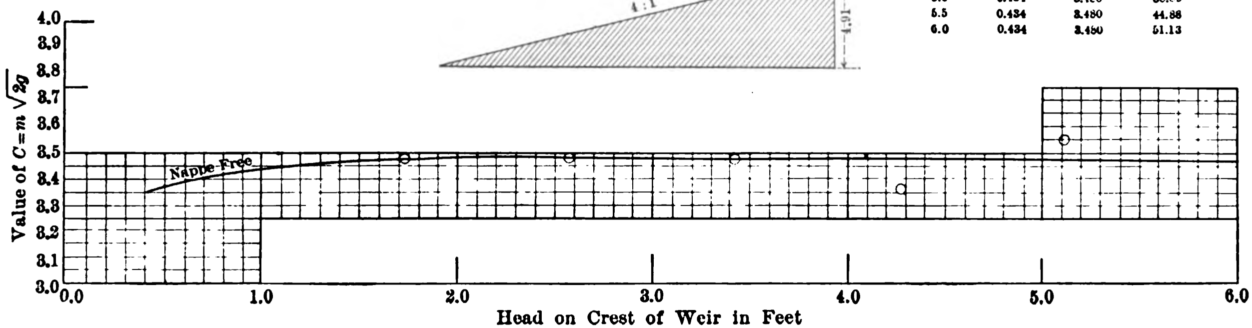
Length of crest 6.58 ft.

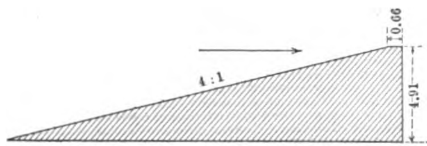
Number of experiments, 3.

Limiting heads, 1.75 ft. and 5.23 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per foot.
0.5	0.412	3.310	1.18
1.0	0.415	3.330	3.33
1.5	0.416	3.340	6.13
2.0	0.418	3.351	9.49
2.5	0.420	3.370	13.32
3.0	0.421	3.380	17.55
3.5	0.422	3.388	22.18
4.0	0.422	3.390	27.12
4.5	0.422	3.390	32.34
5.0	0.422	3.390	37.88
5.5	0.422	3.390	43.71
6.0	0.422	3.390	49.81







CORNELL UNIVERSITY

EXPERIMENT No. 4.

MAY 26TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 6.

Limiting heads, 0.917 ft. and 5.11 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.
0.5			
1.0	0.429	3.440	3.44
1.5	0.432	3.465	6.36
2.0	0.434	3.480	9.83
2.5	0.434	3.485	13.77
3.0	0.434	3.480	18.07
3.5	0.434	3.480	22.77
4.0	0.434	3.480	27.83
4.5	0.434	3.480	33.20
5.0	0.434	3.480	38.89
5.5	0.434	3.480	44.88
6.0	0.434	3.480	51.13

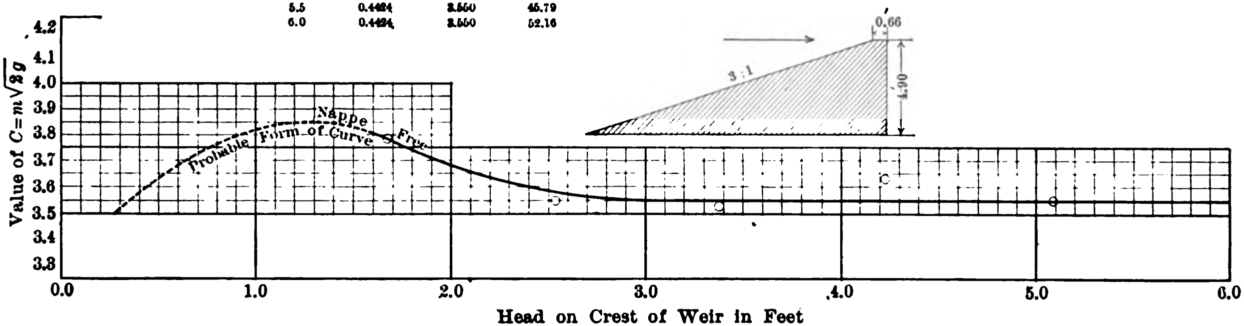
CORNELL UNIVERSITY

EXPERIMENT NO. 5.

MAY 27TH, 1899.

Length of crest 6.56 ft.
Number of experiments, 5.
Limiting heads, 1.689 ft. and 5.085 ft.

Head, in feet.	m	$C = m\sqrt{2g}$	Q per foot.
0.5	0.454	3.64	1.28
1.0	0.476	3.82	3.82
1.5	0.478	3.83	7.17
2.0	0.4596	3.688	10.42
2.5	0.4474	3.590	14.19
3.0	0.4425	3.551	18.46
3.5	0.4424	3.550	23.24
4.0	0.4424	3.550	28.40
4.5	0.4424	3.550	33.87
5.0	0.4424	3.550	39.68
5.5	0.4424	3.550	45.79
6.0	0.4424	3.550	52.16



CORNELL UNIVERSITY

EXPERIMENT No. 6.

BAZIN'S SERIES No. 162.

MAY 27TH, 1899.

Length of crest 6.58 ft.

From full line coefficient curve below

Number of experiments, 5.

Limiting heads, 1.695 ft. and 4.745 ft.

Head, in feet.	m	$C = m\sqrt{2g}$	Q per ft.
0.5	0.5255	4.214	1.50
1.0	0.5288	4.241	4.24
1.5	0.5096	4.088	7.52
2.0	0.4949	3.970	11.22
2.5	0.4848	3.890	15.37
3.0	0.4773	3.830	19.90
3.5	0.4708	3.778	24.75
4.0	0.4661	3.736	29.92
4.5	0.4620	3.708	35.40
5.0	0.4585	3.680	41.14
5.5	0.4548	3.650	47.08
6.0	0.4525	3.630	53.35

CORNELL UNIVERSITY

EXPERIMENT No. 6.

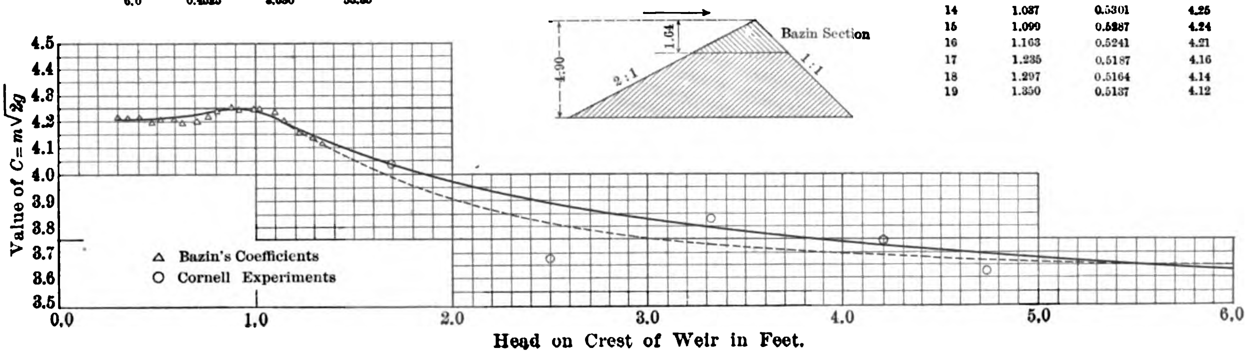
BAZIN'S SERIES No. 162.

MAY 27TH, 1899.

Length of crest 6.58 ft.

Coefficients from dotted curve on coefficient diagram showing effect of a change of position of the mean coefficient curve.

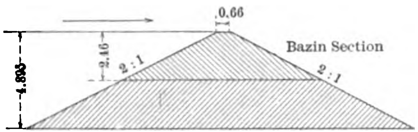
Head, in feet.	m	$C = m\sqrt{2g}$	Q per ft.
0.5	0.525	4.21	1.50
1.0	0.529	4.24	4.24
1.5	0.505	4.05	7.43
2.0	0.486	3.90	11.02
2.5	0.476	3.82	15.10
3.0	0.467	3.75	19.48
3.5	0.464	3.72	24.35
4.0	0.461	3.70	29.60
4.5	0.459	3.68	35.11
5.0	0.458	3.67	41.03
5.5	0.455	3.65	47.06
6.0	0.453	3.65	53.35



BAZIN'S

SERIES No. 162.

No. of experiment.	h=depth on crest, in feet.	m = coefficient of discharge.	$C = m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.302	0.5262	4.22
2	0.360	0.5254	4.22
3	0.418	0.5257	4.22
4	0.472	0.5228	4.20
5	0.529	0.5246	4.21
6	0.586	0.5250	4.21
7	0.644	0.5233	4.20
8	0.697	0.5228	4.20
9	0.756	0.5258	4.22
10	0.810	0.5280	4.24
11	0.866	0.5310	4.26
12	0.928	0.5306	4.25
13	0.982	0.5305	4.25
14	1.037	0.5301	4.25
15	1.099	0.5287	4.24
16	1.163	0.5241	4.21
17	1.235	0.5187	4.16
18	1.297	0.5164	4.14
19	1.350	0.5137	4.12



CORNELL UNIVERSITY

EXPERIMENT No. 8.

BAZIN'S SERIES No. 178.

MAY 30TH, 1899.

Length of crest 6.58 ft.

Upstream face covered with ¼" mesh galvanized wire screening.

Number of experiments, 5.

Limiting heads, 1.657 ft. and 5.011 ft.

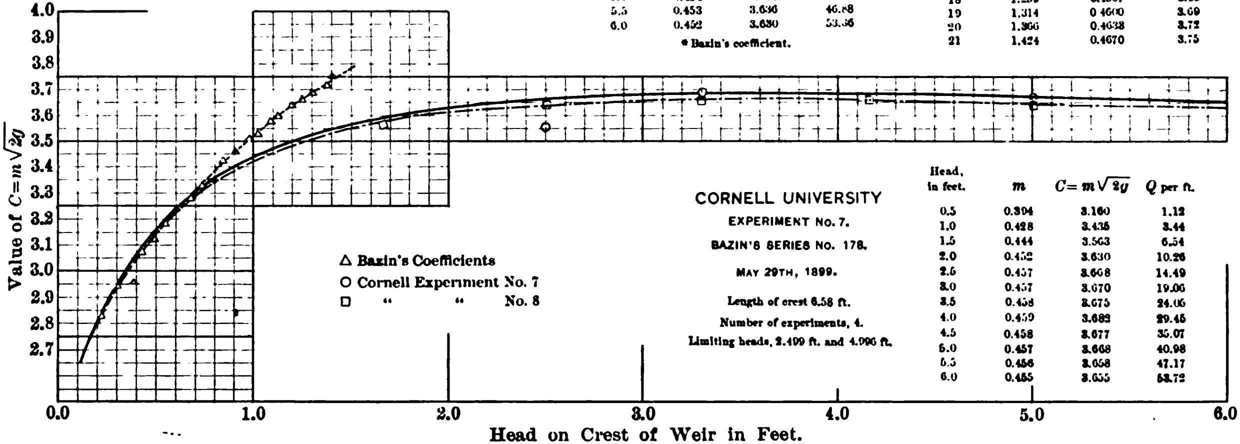
Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.
0.5	0.391	3.140 *	1.11
1.0	0.426	3.422	3.42
1.5	0.442	3.548	6.51
2.0	0.450	3.612	10.21
2.5	0.453	3.640	14.39
3.0	0.456	3.658	18.99
3.5	0.457	3.668	24.00
4.0	0.456	3.662	29.30
4.5	0.455	3.653	34.85
5.0	0.454	3.644	40.74
5.5	0.453	3.636	46.88
6.0	0.452	3.630	53.36

* Bazin's coefficient.

BAZIN'S

SERIES No. 178.

No. of experiment.	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula.
1	0.222	0.3522	2.83
2	0.299	0.3673	2.95
3	0.367	0.3686	2.96
4	0.431	0.3837	3.08
5	0.491	0.3907	3.13
6	0.556	0.3961	3.19
7	0.614	0.4038	3.24
8	0.669	0.4093	3.28
9	0.732	0.4147	3.33
10	0.789	0.4190	3.36
11	0.847	0.4271	3.43
12	0.906	0.4315	3.46
13	0.966	0.4379	3.51
14	1.028	0.4398	3.53
15	1.063	0.4467	3.58
16	1.142	0.4493	3.60
17	1.196	0.4540	3.64
18	1.259	0.4567	3.66
19	1.314	0.4600	3.69
20	1.366	0.4638	3.72
21	1.424	0.4670	3.75



CORNELL UNIVERSITY

EXPERIMENT No. 7.

BAZIN'S SERIES No. 178.

MAY 29TH, 1899.

Length of crest 6.58 ft.

Number of experiments, 4.

Limiting heads, 2.499 ft. and 4.996 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.
0.5	0.394	3.160	1.12
1.0	0.428	3.435	3.44
1.5	0.444	3.563	6.54
2.0	0.452	3.630	10.26
2.5	0.457	3.668	14.49
3.0	0.457	3.670	19.06
3.5	0.458	3.675	24.06
4.0	0.459	3.682	29.45
4.5	0.458	3.677	35.07
5.0	0.457	3.668	40.98
5.5	0.456	3.658	47.17
6.0	0.455	3.655	53.72

CORNELL UNIVERSITY

EXPERIMENT NO. 9.

BAZIN'S SERIES NO. 173.

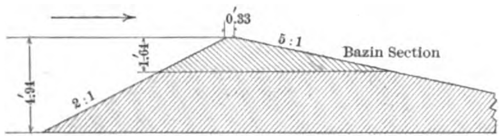
MAY 31ST, 1899.

Length of crest 6.56 ft.

Number of experiments, 4.

Limiting heads, 1.677 ft. and 4.848 ft.

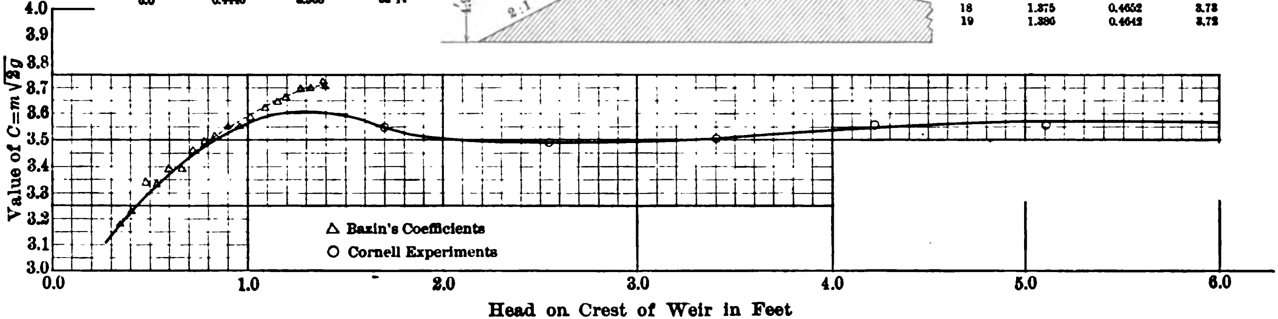
Head, in feet.	m	$C=m\sqrt{2g}$	Q per foot.
0.5	0.4112	3.300	1.17
1.0	0.4450	3.570	3.57
1.5	0.4480	3.596	6.60
2.0	0.4374	3.510	9.92
2.5	0.4355	3.495	13.83
3.0	0.4356	3.496	18.16
3.5	0.4375	3.510	22.97
4.0	0.4410	3.537	28.30
4.5	0.4436	3.560	33.96
5.0	0.4446	3.568	39.87
5.5	0.4446	3.568	46.01
6.0	0.4446	3.568	52.44



BAZIN'S

SERIES NO. 173.

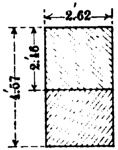
No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C=m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.346	0.3962	3.18
2	0.410	0.4030	3.23
3	0.471	0.4159	3.34
4	0.532	0.4155	3.33
5	0.594	0.4228	3.39
6	0.658	0.4233	3.39
7	0.714	0.4319	3.46
8	0.769	0.4363	3.50
9	0.834	0.4398	3.52
10	0.892	0.4438	3.56
11	0.960	0.4447	3.56
12	1.021	0.4480	3.59
13	1.074	0.4532	3.63
14	1.137	0.4555	3.65
15	1.196	0.4572	3.67
16	1.260	0.4610	3.70
17	1.315	0.4619	3.71
18	1.375	0.4652	3.73
19	1.386	0.4642	3.72



BAZIN'S

SERIES NO. 114.

No. of experiment.	h=depth on crest. In feet.	m=coefficient of discharge.	$m\sqrt{2g}$= coefficient for Francis's formula.
1	0.204	0.3078	2.47
2	0.280	0.3167	2.54
3	0.352	0.3229	2.59
4	0.433	0.3246	2.00
5	0.504	0.3233	2.59
6	0.578	0.3241	2.60
7	0.657	0.3270	2.62
8	0.735	0.3272	2.63
9	0.810	0.3281	2.63
10	0.888	0.3300	2.65
11	0.958	0.3316	2.66
12	1.034	0.3336	2.68
13	1.112	0.3351	2.69
14	1.171	0.3361	2.70
15	1.243	0.3398	2.73
16	1.301	0.3407	2.73
17	1.384	0.3440	2.76



Bazin's Section

CORNELL UNIVERSITY

EXPERIMENT NO. 10. JUNE 1ST, 1899

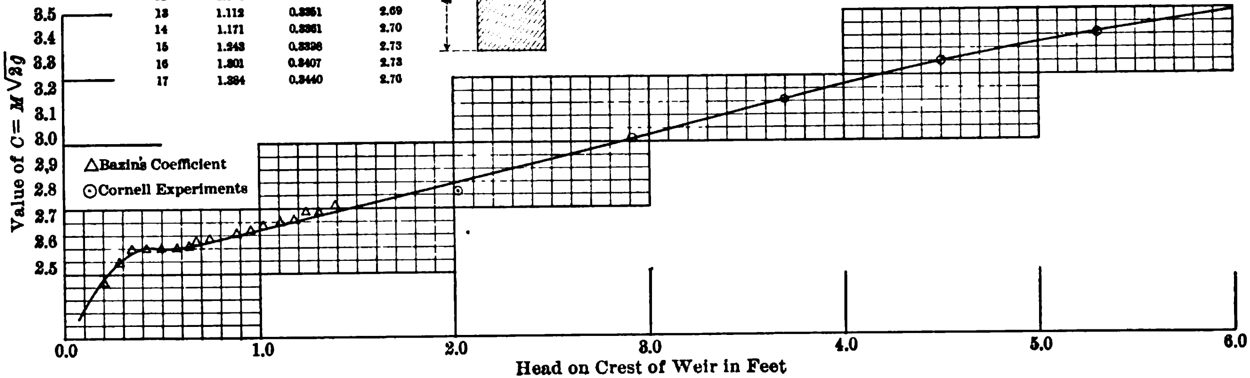
Length of crest 6.56 ft.

Number of experiments, 5.

Limiting heads, 2.038 ft. and 5.284 ft.

Head. In feet. (1)	m (2)	$C=m\sqrt{2g}$ (3)	Q per ft. (4)
0.5	0.324	2.600*	0.93
1.0	0.333	2.672*	2.67
1.5	0.343	2.750*	5.04
2.0	0.354	2.840	8.03
2.5	0.365	2.928	11.53
3.0	0.376	3.015	15.66
3.5	0.388	3.113	20.37
4.0	0.400	3.208	25.67
4.5	0.411	3.297	31.45
5.0	0.422	3.388	37.80
5.5	0.426	3.420	44.10
6.0	0.433	3.480	51.15

* Bazin's coefficients.



BAZIN'S

SERIES NO. 116.

No. of experiment.	h=depth on crest, in feet.	m=coefficient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula.
1	0.177	0.3372	2.71
2	0.225	0.3526	2.83
3	0.296	0.3607	2.90
4	0.357	0.3637	2.92
5	0.425	0.3679	2.95
6	0.504	0.3707	2.98
7	0.537	0.3731	2.99
8	0.639	0.3753	3.01
9	0.718	0.3734	3.00
10	0.781	0.3742	3.00
11	0.849	0.3764	3.02
12	0.917	0.3765	3.02
13	0.986	0.3797	3.05
14	1.063	0.3813	3.06
15	1.120	0.3841	3.08
16	1.186	0.3850	3.09
17	1.251	0.3863	3.10
18	1.317	0.3888	3.12

CORNELL UNIVERSITY

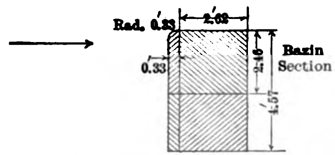
EXPERIMENT NO. 11.

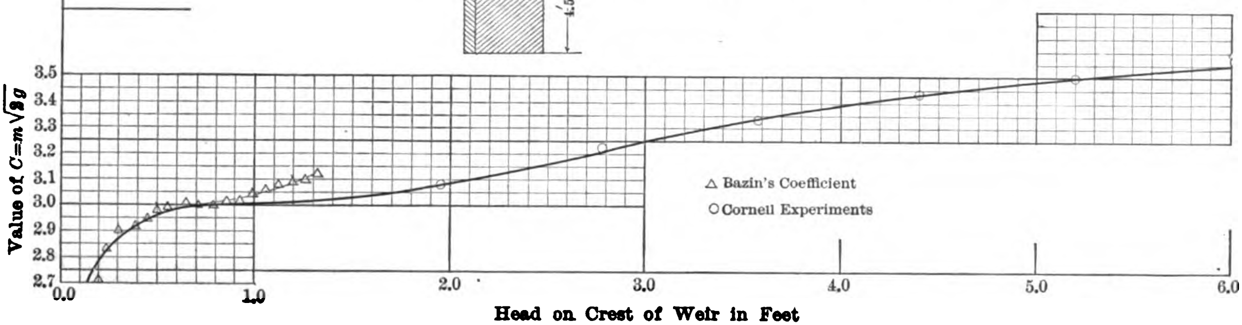
BAZIN'S SERIES NO. 116.

JUNE 1ST, 1899.

Length of crest 6.56 ft.
Number of experiments, 5.
Limiting heads, 1.949 ft. and 5.208 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per foot.
0.5	0.369	2.960	1.06
1.0	0.375	3.008	3.01
1.5	0.378	3.033	5.57
2.0	0.384	3.086	8.72
2.5	0.388	3.115	12.31
3.0	0.405	3.252	16.89
3.5	0.414	3.326	21.77
4.0	0.422	3.385	27.08
4.5	0.428	3.436	32.77
5.0	0.433	3.475	38.84
5.5	0.437	3.510	45.27
6.0	0.442	3.549	52.70





CORNELL UNIVERSITY

EXPERIMENT NO. 12.

BAZIN'S SERIES NO. 115.

JUNE 20, 1899.

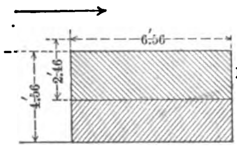
Length of crest 6.58 ft.

Number of experiments, 4.

Limiting heads, 2.175 ft. and 5.102 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per foot.
0.5	0.3117	2.502 *	0.90
1.0	0.3238	2.600	2.60
1.5	0.3170	2.545	4.67
2.0	0.3096	2.485	7.04
2.5	0.3088	2.480	9.80
3.0	0.3133	2.515	13.06
3.5	0.3198	2.563	16.78
4.0	0.3251	2.610	20.88
4.5	0.3307	2.655	25.35
5.0	0.3364	2.700	30.18
5.5	0.3414	2.740	35.33
6.0	0.3437	2.760	40.56

* Bazin's coefficient.

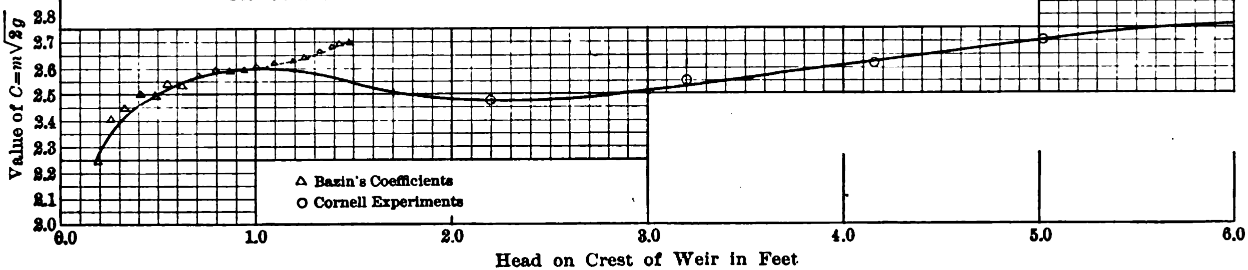


Bazin Section

BAZIN'S

SERIES NO. 115.

No. of experi-ment.	h=depth on crest, in feet.	m=coeffi-cient of discharge.	$C=m\sqrt{2g}$= coefficient for Francis' formula.
1	0.196	0.2747	2.25
2	0.264	0.3014	2.41
3	0.342	0.3047	2.45
4	0.415	0.3123	2.51
5	0.495	0.3119	2.50
6	0.566	0.3184	2.55
7	0.688	0.3178	2.54
8	0.716	0.3194	2.56
9	0.792	0.3243	2.00
10	0.871	0.3245	2.00
11	0.948	0.3246	2.60
12	1.023	0.3250	2.61
13	1.097	0.3281	2.63
14	1.178	0.3294	2.64
15	1.200	0.3302	2.65
16	1.330	0.3342	2.68
17	1.388	0.3348	2.69
18	1.424	0.3365	2.70
19	1.467	0.3362	2.70



CORNELL UNIVERSITY

EXPERIMENT NO. 13

BAZIN'S SERIES NO. 117

JUNE 2D, 1899.

Length of crest 6.56 ft.

Number of experiments, 5.

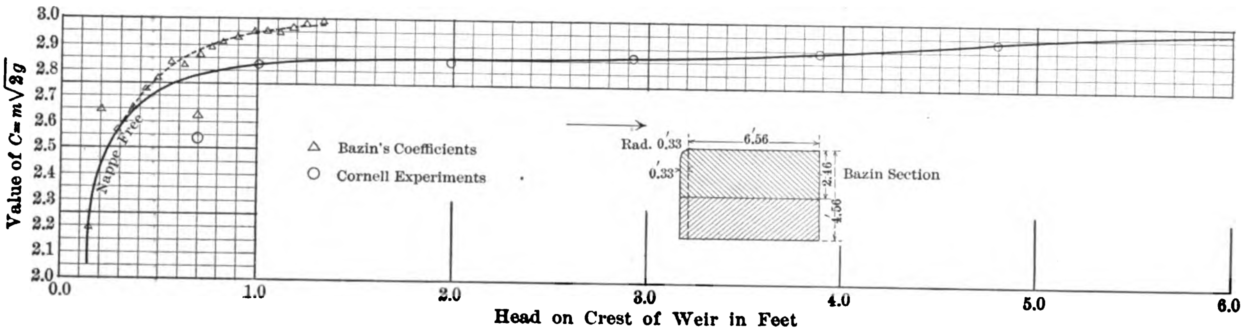
Limiting heads, 1.016 ft. and 4.880 ft.

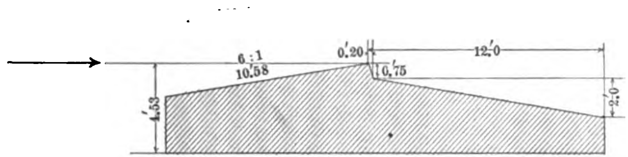
Head, in feet.	m	$C=m\sqrt{2g}$	Q per foot.
0.5	0.338	2.716	0.98
1.0	0.352	2.823	2.83
1.5	0.354	2.842	5.22
2.0	0.354	2.844	8.06
2.5	0.355	2.850	11.26
3.0	0.356	2.860	14.85
3.5	0.358	2.877	18.83
4.0	0.360	2.896	23.17
4.5	0.363	2.918	27.84
5.0	0.367	2.945	32.91
5.5	0.369	2.963	38.22
6.0	0.370	2.973	43.70

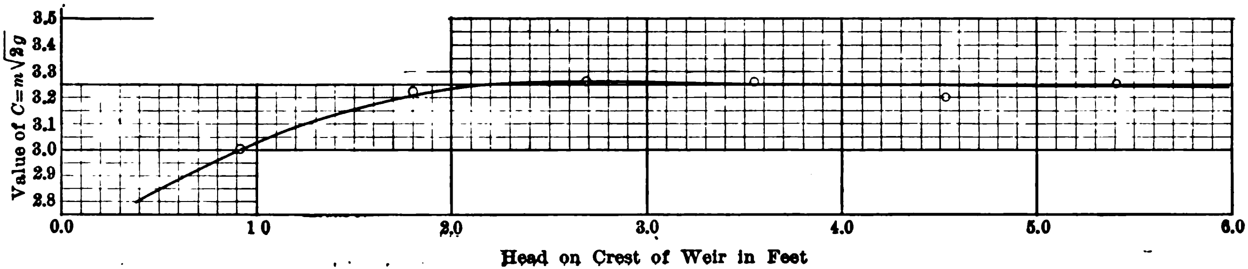
BAZIN'S

SERIES NO. 117

No. of experiment.	h = depth on crest, in feet.	m = coefficient of discharge.	$C=m\sqrt{2g}$ = coefficient for Francis' formula.
1	0.158	0.2741	2.19
2	0.204	0.3287	2.64
3	0.289	0.3207	2.57
4	0.361	0.3286	2.65
5	0.426	0.3396	2.73
6	0.494	0.3452	2.77
7	0.562	0.3533	2.83
8	0.635	0.3521	2.82
9	0.708	0.3563	2.86
10	0.771	0.3604	2.89
11	0.834	0.3629	2.91
12	0.912	0.3649	2.93
13	0.989	0.3671	2.95
14	1.064	0.3680	2.95
15	1.129	0.3678	2.95
16	1.197	0.3701	2.97
17	1.267	0.3712	2.98
18	1.336	0.3731	2.99







CORNELL UNIVERSITY

EXPERIMENT NO. 14.

REXFORD FLATS DAM.

JUNE 3RD, 1899.

Length of crest 6.58 ft.

Number of experiments, 6.

Limiting heads, 0.924 ft. and 5.418 ft.

Head, in feet.	m	$C = m\sqrt{2g}$	Q per foot.
0.5	0.356	2.850	1.02
1.0	0.377	3.025	3.02
1.5	0.394	3.160	5.80
2.0	0.403	3.232	9.15
2.5	0.406	3.255	12.86
3.0	0.406	3.255	16.90
3.5	0.405	3.250	21.27
4.0	0.405	3.250	26.00
4.5	0.404	3.245	30.97
5.0	0.404	3.242	36.22
5.5	0.403	3.238	41.70
6.0	0.403	3.230	47.56

CORNELL UNIVERSITY

EXPERIMENT No. 16.

REXFORD FLATS DAM.

With up-stream edge of crest rounded to a radius of 0.328 ft.

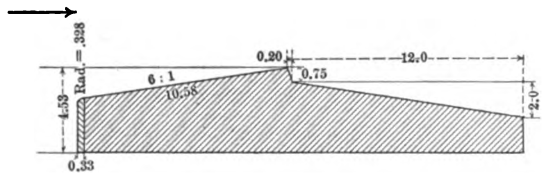
JUNE 30, 1899.

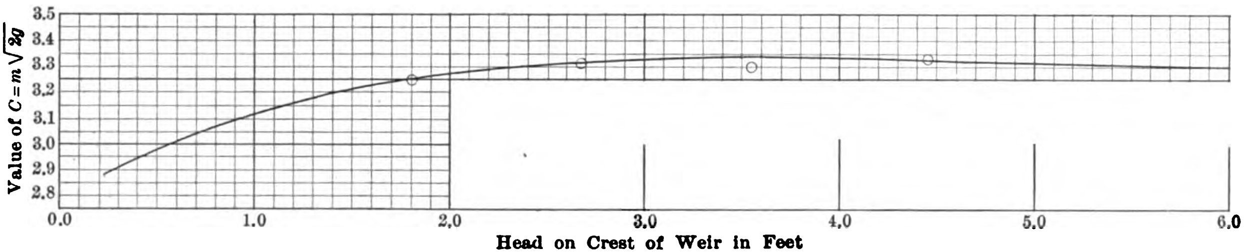
Length of crest 6.56 ft.

Number of experiments, 4.

Limiting heads, 1.786 ft. and 4.439 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.
0.5	0.371	2.980	1.06
1.0	0.389	3.120	3.12
1.5	0.400	3.210	5.90
2.0	0.408	3.270	9.25
2.5	0.412	3.306	13.07
3.0	0.415	3.332	17.32
3.5	0.416	3.340	21.86
4.0	0.415	3.331	26.65
4.5	0.414	3.325	31.72
5.0	0.413	3.315	37.15
5.5	0.412	3.306	42.65
6.0	0.411	3.300	48.48





CORNELL UNIVERSITY

EXPERIMENT No. 16.

LITTLE FALLS DAM, SECTION No. 1.

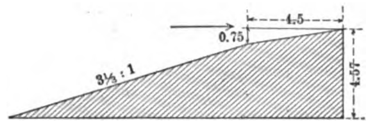
JUNE 5TH, 1899.

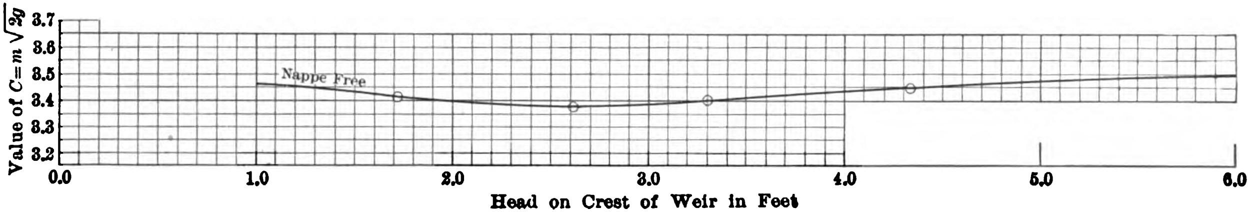
Length of crest 6.58 ft.

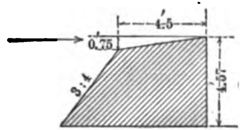
Number of experiments, 4.

Limiting heads, 1.736 ft. and 4.339 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.
0.5			
1.0	0.432	3.468	3.47
1.5	0.428	3.434	6.30
2.0	0.424	3.400	9.62
2.5	0.421	3.380	13.36
3.0	0.422	3.386	17.58
3.5	0.425	3.411	22.32
4.0	0.428	3.436	27.48
4.5	0.431	3.460	33.01
5.0	0.433	3.478	38.87
5.5	0.435	3.490	45.00
6.0	0.436	3.500	51.44







CORNELL UNIVERSITY

EXPERIMENT NO. 17

LITTLE FALLS DAM, SECTION NO. 2.

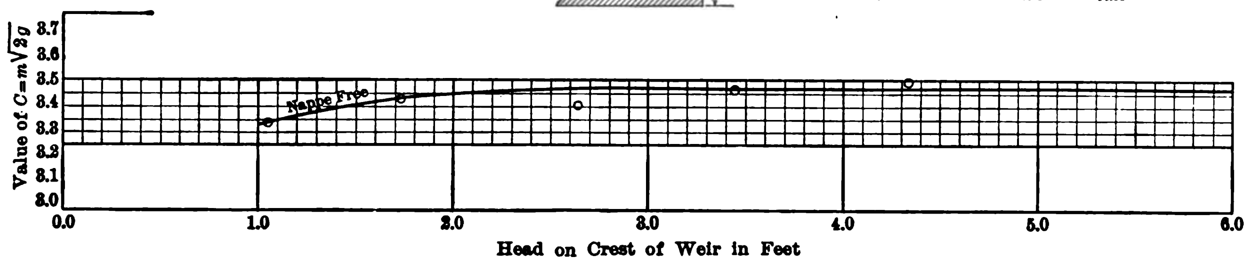
JUNE 5TH, 1899.

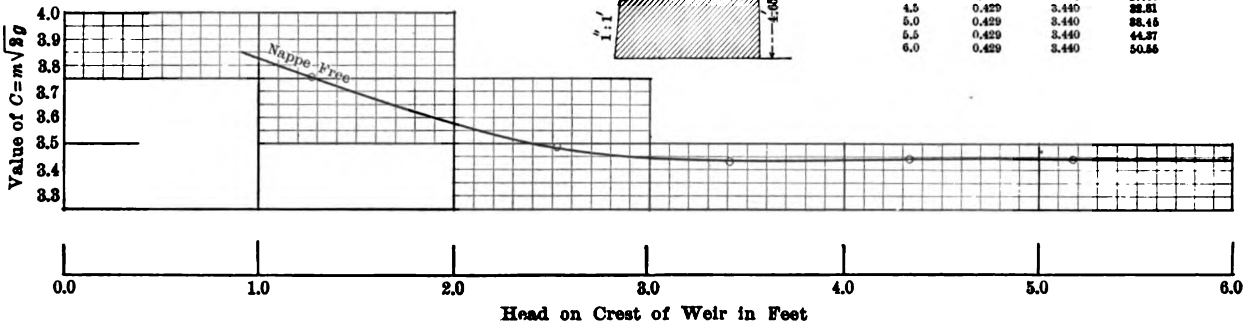
Length of crest 6.56 ft.

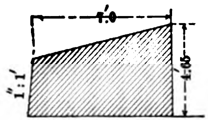
Number of experiments, 5.

Limiting heads, 1.067 ft. and 4.336 ft.

Head, in feet.	m	$C = m\sqrt{2g}$	Q per foot.
0.5			
1.0	0.415	3.330	3.33
1.5	0.425	3.408	6.26
2.0	0.430	3.450	9.76
2.5	0.432	3.465	13.70
3.0	0.432	3.470	18.08
3.5	0.432	3.465	22.66
4.0	0.432	3.465	27.75
4.5	0.432	3.465	33.06
5.0	0.432	3.465	38.75
5.5	0.432	3.465	44.68
6.0	0.432	3.465	50.90







CORNELL UNIVERSITY

EXPERIMENT NO. 18.

SECTION OF SPILLWAY OF INDIAN LAKE DAM.

JUNE 6TH, 1899.

Length of crest 6.56 ft.

Number of experiments, 5.

Limiting heads, 1.277 ft. and 5.180 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per foot.
0.5			
1.0	0.476	3.820	3.82
1.5	0.460	3.695	6.78
2.0	0.458	3.675	10.38
2.5	0.436	3.498	13.79
3.0	0.429	3.445	17.89
3.5	0.427	3.430	22.46
4.0	0.428	3.434	27.46
4.5	0.429	3.440	32.81
5.0	0.429	3.440	38.45
5.5	0.429	3.440	44.37
6.0	0.429	3.440	50.56

CORNELL UNIVERSITY

EXPERIMENT No. 19.

SECTION FOR SUBMERGED WEIR.

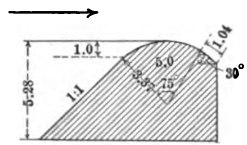
JUNE 7TH, 1899.

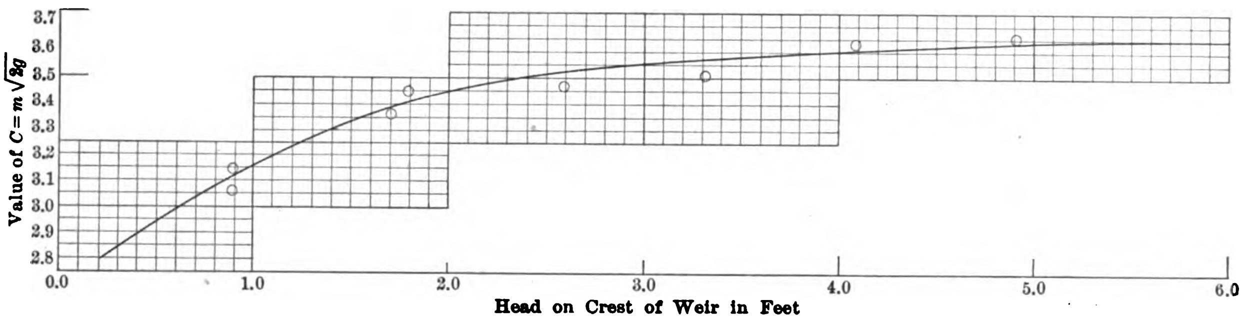
Length of crest 6.56 ft.

Number of experiments, 8

Limiting heads, 0.367 and 4.918 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.
0.5	0.367	2.945	1.04
1.0	0.394	3.165	3.17
1.5	0.412	3.306	6.08
2.0	0.430	3.454	9.77
2.5	0.439	3.520	13.91
3.0	0.443	3.555	18.48
3.5	0.447	3.586	23.46
4.0	0.450	3.610	28.88
4.5	0.452	3.630	34.63
5.0	0.455	3.650	40.80
5.5	0.457	3.666	47.28
6.0	0.458	3.675	54.02





On April 14th, 1899, the writer visited the new Cornell University Hydraulic Laboratory and at once saw that a fine opportunity was offered there to experiment on flows over weirs at much higher heads than had hitherto been possible; and on communicating his views to the United States Board of Engineers on Deep Waterways, he was permitted to undertake a series of experiments in co-operation with the University authorities. Messrs. Wallace Greenalch, Assoc. M. Am. Soc. C. E., Robert E. Horton, and George E. Cook were detailed from the Deep Waterways engineering corps for this work, which was done in co-operation with Professor Gardner S. Williams, M. Am. Soc. C. E., Engineer in Charge of the Hydraulic Laboratory, permission to use the same having been obtained by correspondence with Professor E. A. Fuertes, M. Am. Soc. C. E., Director of Cornell University College of Civil Engineering. The writer gave the experiments general supervision, the working out of the details being mostly left to Mr. Greenalch and Professor Williams, Mr. Greenalch undertaking to construct the necessary flumes, bulkheads, experimental weirs, etc., and Professor Williams preparing and taking charge of the measuring apparatus. The reductions were made by Messrs. Greenalch, Horton and Cook under the direction of the writer.

Fig. 1 is a plan and section of the experimental channel at the Cornell University Hydraulic Laboratory. This laboratory has been quite fully described in *Engineering News* for March 2d, 1899, and no farther description will be given here than is necessary to explain the experiments.

Description of the Arrangements for the Experiments.—The canal in which the experiments were made consists, briefly, of a channel with sides and bottom of concrete. It is 418 ft. long, 16 ft. wide and 10 ft. deep. The gradient of the bottom of the channel is at the rate of 1 ft. in 500. A bulkhead composed of 12 x 12-in. timbers, situated about 60 ft. from the upper end, divides the channel into two chambers. A standard sharp-edged weir, 16 ft. in length, was placed on this bulkhead, the crest of the weir being 13.13 ft. above the bottom of the channel. The upper chamber above the bulkhead has higher side walls than the lower chamber, which admitted of a depth of 17.7 ft. of water. At the lower end of the channel another timber bulkhead closes the lower chamber, and on this the weirs to be experimented upon were built. The top of this bulkhead was about 4.8 ft. above the

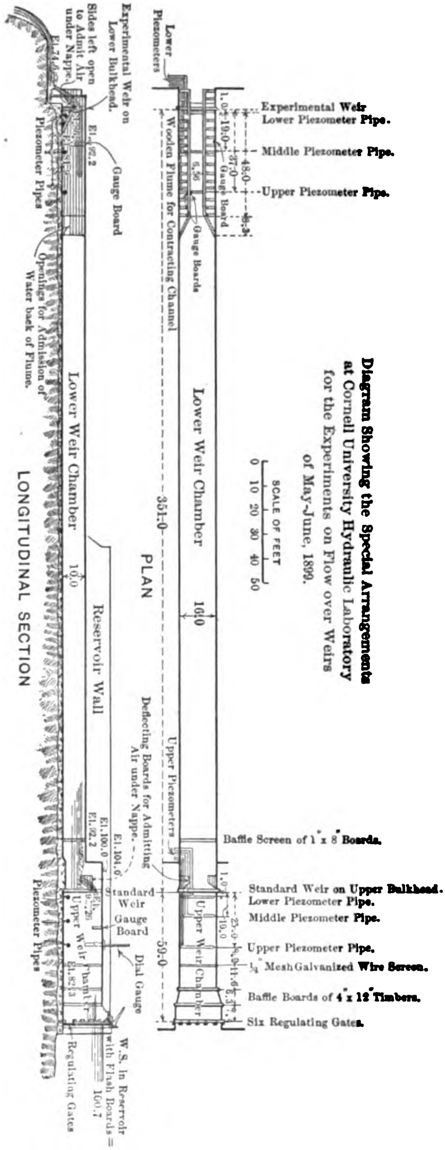


Diagram Showing the Special Arrangements
at Cornell University Hydraulic Laboratory
for the Experiments on Flow over Weirs
of May-June, 1899.
SCALE OF FEET
0 10 20 30 40 50
Experimental Weir
Lower Piezometer Pipe.
Middle Piezometer Pipe.
Upper Piezometer Pipe.
Baffle Screen of 1" x 8" Boards.
Standard Weir on Upper Bulkhead.
Lower Piezometer Pipe.
Middle Piezometer Pipe.
Upper Piezometer Pipe.
¼" Mesh Galvanized Wire Screen.
Baffle Boards of 4" x 12" Timbers.
Six Regulating Gates.
Lower Piezometers
Wooden Flume for Contracting Channel
Gauge Board
Gauge Boards
Lower Weir Chamber
351-0
PLAN
Upper Piezometers
Deflecting Boards for Admitting Air under Nappe.
Upper Weir Chamber
59-0
Experimental Weir on Lower Bulkhead.
Sides left open to Admit Air under Nappe.
Piezometer Pipes
Openings for Admission of Water back of Flume.
Lower Weir Chamber
Reservoir Wall
LONGITUDINAL SECTION
Standard Weir
Gauge Board
Dial Gauge
W.S. in Reservoir with Flash Boards = 100.7
Regulating Gates
Piezometer Pipes

FIG. 1

bottom of the channel. The heights vary slightly for each experimental weir, the exact height of each being shown on the sections at the head of the tabulations of results.

In order to obtain heads of about 5 ft. on the lower weir the 16-ft. channel was narrowed to a width of 6.56 ft. (2 m.) by means of a wooden flume, as shown in Fig. 1. This flume was 6.56 ft. wide for a distance of 48 ft. above the lower bulkhead and then expanded to a width of 16 ft. in a length of 8.3 ft., as shown. The flume was constructed of matched, white pine boards 1.75 ins. thick, planed on the inside and held in place by bents of 4 by 4-in. timbers. As the lower bulkhead was water-tight for the whole width of the channel, no attempt was made to construct the sides and bottoms of the flume absolutely water-tight, although they were practically so. Inasmuch as nearly equal pressure on both sides of flume would permit of greater economy in construction, two boards were left off each side at the upper end, thus allowing the water to enter at the sides between the flume and the concrete walls of the main canal. This arrangement also diminished greatly the area of water-tight work. The sides of the flume were extended from 8 to 24 ft. below the bulkhead according to the form of weir experimented upon, thus preventing lateral expansion of the nappe after passing the weir and crest. Openings were left in each side of this extension below the level of the crest in order to certainly allow free access of air under the nappe. The vertical fall of the water from the crest of the experimental weir to the rock below was about 12.2 ft.

There is a pond of 22 acres above the main reservoir dam, the surface of which was raised about 1.7 ft. by means of flash boards placed on the main spillway. The water surface thus obtained was about 5.4 ft. above the crest of the standard weir. Water from the reservoir was admitted into the upper chamber through six wooden sluice gates, operated by rack and pinion apparatus with long levers.

The sharp-edged standard weir was composed of a 3.5 by 5-in. steel angle, secured by lag screws to a 6 by 12-in. oak timber, as shown in Fig. 1. After bolting the angle-iron into place on the timber, the edge of the 5-in. leg was planed and dressed to a true line $\frac{1}{16}$ in. in width, and carefully leveled in position on the upper bulkhead. Air was admitted freely under the nappe by means of deflecting boards at each end of the weir. The fall of the water from the crest of the standard

weir to the water surface in the lower chamber varied from about 3 to 8 ft., according to the quantity of water flowing.

The velocity of the water passing through the regulating gates at the extreme head of the channel was checked by three screens in the upper chamber. The first two consisted of 4 by 12-in. timbers placed horizontally with the wide face toward the current, and spaced from 8 to 12 ins. apart. Below these was a third screen of ¼ in. mesh, galvanized wire netting. A screen, composed of 1 in. by 8 in. boards, laid horizontally with the edges to the current and spaced 2 ins. apart, was placed about 20 ft. below the upper bulkhead, and served to quiet the water in the lower chamber.

The heads on the weirs were measured by means of piezometers, constructed as follows: A 1-in. galvanized iron pipe, with holes ¼ in. in diameter and spaced 6 ins. apart, was laid across the channel about 8 ins. above the bottom, with the holes therein opening downward. Connections with these pipes were made by ¾-in. pipes passing through the bulkhead to a point below the weir, where the gauges could easily be connected by rubber hose. The gauges were glass tubes, ¾ in. internal diameter, mounted on wooden standards, and read by a scale graduated to 2-mm. spaces. Three piezometers were set at each weir, as shown in Fig. 1, though readings were taken only on the upper two. In order to check the accuracy of the piezometric readings, at the conclusion of Experiment No. 17, a fourth piezometer pipe was set in the bottom of the flume above the lower bulkhead, and about 6 ins. up-stream from the upper piezometer. This pipe was set with ¼-in. holes directly on top, and with the top of the pipe flush with the bottom of the flume. Readings were taken simultaneously on both piezometers, and considerable differences noted.

At the lower weir the height of the flowing water in the flume and of the still water behind the flume was read on scales marked on the side of the flume. These scales were divided to 0.05 ft. and read by interpolation to 0.01 ft. Similar gauges were set in the upper chamber and in the reservoir, and readings of each were taken every five minutes.

In addition to the gauge board in the upper chamber, a float gauge, read by dial to 0.01 ft., was set directly over the upper piezometer. The float of this dial gauge was a heavy, sheet-tin, air-tight vessel, weighted with shot and caused to move vertically with the water in

the interior of a length of 8-in. cast-iron pipe, suspended from two timbers across the upper channel. The dial was placed in such a position as to be read easily by the assistants operating the gates at the upper end of the channel, thus insuring that the depth over the standard weir be easily maintained at substantially a constant head, as required.

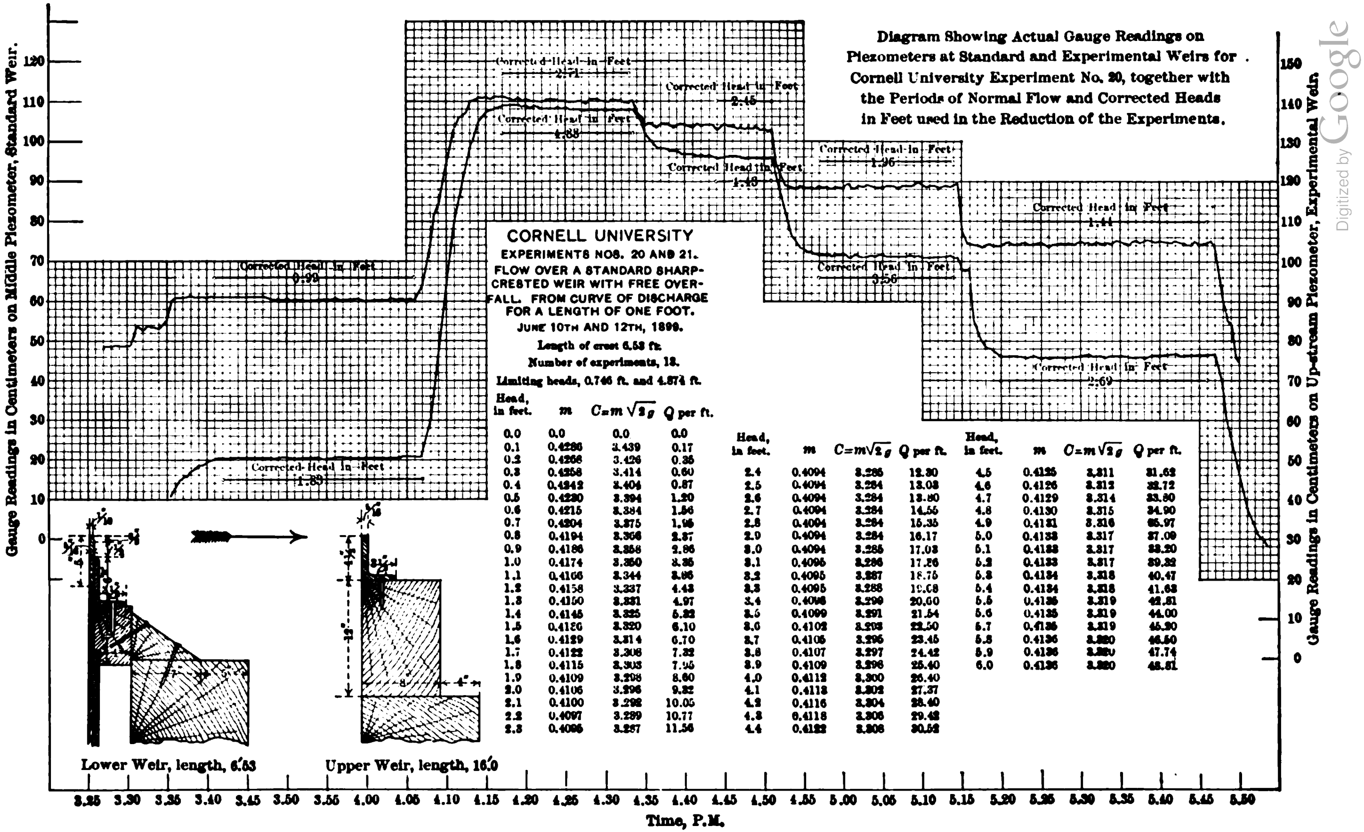
The experimental weirs of different sections were all made of planed, white pine boards about 1 in. thick, so fastened together with screws as to be easily removable. Leakage from the bulkheads and gates at the lower end of the channel was effectively stopped by caulking with oakum and pitch, and also by the application of wheat bran from time to time.

Observations showed that the upper bulkhead was practically water-tight, but at the side gates, near the lower bulkhead, there was a slight leakage, which has been taken at 0.5 cu. ft. per second for high heads and 0.25 cu. ft. per second for low heads, and proportionally between. The main channel, with side walls of concrete, is considered to be water-tight.

The foregoing allowances for leakage are taken to cover the slight evaporation and absorption loss into the sides of the main channel.

Reduction of the Experiments.—The method of conducting the experiments was, in general, as follows: The main head gates at the entrance to the canal were open to such a width that the dial gauge showed a head of 3 ft. above the standard 16-ft weir. They were retained in this position until a uniform regimen of flow was established in the canal and maintained for a period of from 10 to 20 minutes, during which time the piezometers were read at both weirs at intervals of 30 seconds. At the close of such a period the head gates were lowered until the dial gauge showed a reading of 2.5 ft. head on the upper weir, sufficient time being allowed to elapse to establish and maintain a new regimen of flow, the same as before. In this way piezometric observations were taken during several periods at different heads in each experiment, usually terminating with a head or about 6 ins. or 1 ft. on the upper weir. In some cases the varying of the head on the upper weir by uniform decrements of 6 ins. was not adhered to.

The method of treating the piezometric readings, obtained as described in the preceding paragraph, is shown in Figs. 2 and 3. In Fig. 2 the upper curve shows the readings taken at the upper weir (16 ft.



CORNELL UNIVERSITY
EXPERIMENTS NOS. 20 AND 21.
FLOW OVER A STANDARD SHARP-CRESTED WEIR WITH FREE OVER-FALL. FROM CURVE OF DISCHARGE FOR A LENGTH OF ONE FOOT.
JUNE 10TH AND 12TH, 1899.
Length of crest 6.53 ft.
Number of experiments, 13.
Limiting heads, 0.746 ft. and 4.874 ft.

Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.	Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.	Head, in feet.	m	$C=m\sqrt{2g}$	Q per ft.
0.0	0.0	0.0	0.0								
0.1	0.4286	3.439	0.17								
0.2	0.4268	3.426	0.35								
0.3	0.4256	3.414	0.60	2.4	0.4094	3.285	12.30	4.5	0.4125	3.311	31.62
0.4	0.4242	3.404	0.87	2.5	0.4094	3.284	13.03	4.6	0.4126	3.312	32.72
0.5	0.4230	3.394	1.20	2.6	0.4094	3.284	13.80	4.7	0.4129	3.314	33.80
0.6	0.4215	3.384	1.56	2.7	0.4094	3.284	14.55	4.8	0.4130	3.315	34.90
0.7	0.4204	3.375	1.95	2.8	0.4094	3.284	15.35	4.9	0.4131	3.316	35.97
0.8	0.4194	3.366	2.37	2.9	0.4094	3.284	16.17	5.0	0.4133	3.317	37.09
0.9	0.4186	3.358	2.86	3.0	0.4094	3.285	17.03	5.1	0.4133	3.317	38.20
1.0	0.4174	3.350	3.35	3.1	0.4095	3.286	17.86	5.2	0.4133	3.317	39.32
1.1	0.4166	3.344	3.86	3.2	0.4095	3.287	18.75	5.3	0.4134	3.318	40.47
1.2	0.4158	3.337	4.43	3.3	0.4095	3.288	19.68	5.4	0.4134	3.318	41.63
1.3	0.4150	3.331	4.97	3.4	0.4096	3.290	20.60	5.5	0.4135	3.319	42.81
1.4	0.4145	3.325	5.32	3.5	0.4099	3.291	21.54	5.6	0.4135	3.319	44.00
1.5	0.4130	3.320	6.10	3.6	0.4102	3.293	22.50	5.7	0.4135	3.319	45.20
1.6	0.4129	3.314	6.70	3.7	0.4105	3.295	23.45	5.8	0.4136	3.320	46.50
1.7	0.4122	3.306	7.32	3.8	0.4107	3.297	24.42	5.9	0.4136	3.320	47.74
1.8	0.4115	3.303	7.95	3.9	0.4109	3.298	25.40	6.0	0.4136	3.320	48.81
1.9	0.4109	3.298	8.60	4.0	0.4112	3.300	26.40				
2.0	0.4106	3.296	9.32	4.1	0.4113	3.302	27.37				
2.1	0.4100	3.292	10.05	4.2	0.4116	3.304	28.40				
2.2	0.4097	3.289	10.77	4.3	0.4118	3.306	29.42				
2.3	0.4096	3.287	11.56	4.4	0.4122	3.308	30.52				

FIG. 2.

in length), and the lower curve the readings taken at the lower experimental weir for Experiment No. 14 on the Rexford Flats dam. These two curves, as plotted, show the actual readings taken, in centimeters, without corrections of any sort or kind. The several periods (*A–A*) of the upper curve, and the corresponding periods (*B–B*) of the lower curve represent the actual periods taken in the reductions for each height experimented upon. A mean of the actual readings for these periods has been taken as the mean head during each experimental period.

Fig. 3 is a plotting of similar curves for Experiment No. 20 on a sharp-crested weir, made on June 10th, 1899. The explanations for Fig. 2 apply equally to this figure. Attention may be directed to the method of exhibiting the continuous curve of flow, as shown in Figs. 2 and 3. Its use in the present case is to be credited to Professor Williams. The writer has never seen it used before, and, if it is original with Professor Williams, he is entitled to very great credit for this particular feature of the experiments.

In order to calibrate the upper weir (16 ft. in length), and thereby determine the quantity of water flowing in the lower canal and over the experimental weirs, Experiments Nos. 20 and 21 were made. These experiments apply to a sharp-crested weir of standard form, 5.26 ft. in height, placed in the lower end of the canal. As a basis for the reduction of Experiments Nos. 20 and 21 a discharge curve has been computed for the upper weir for heads up to 0.6 m. (1.969 ft.), using Bazin's formula:

$$Q = M\,L\,H\sqrt{2\,g\,H}, \text{ and}$$

$$M = u\left[l + 0.55\left(\frac{H}{P+H}\right)^2\right]$$

where

Q = discharge over weir, in cubic feet per second;

l = length of crest, in feet;

H = head on crest, in feet;

M = coefficient of discharge;

P = height of crest of weir on up-stream side above bottom of channel of approach, in feet; and

u = a coefficient which depends upon the height of the weir and which has been taken from Bazin's table.*

* See *Annales des Ponts et Chaussées, Mémoires et Documents*, 1888, p. 446.

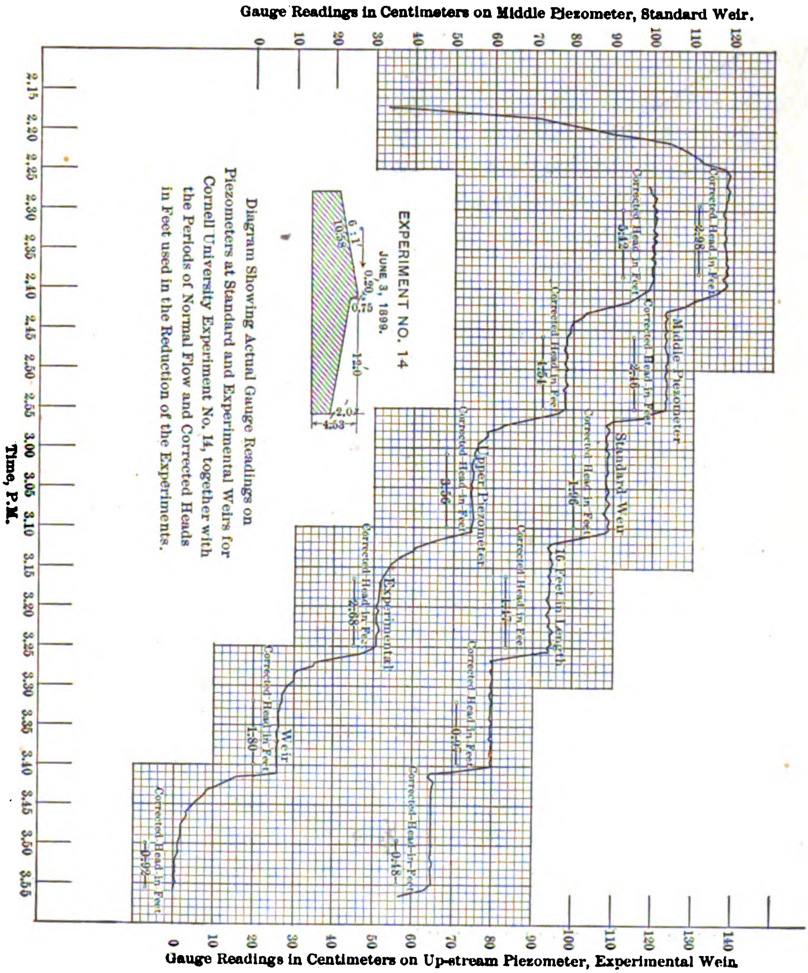

Gauge Readings in Centimeters on Middle Piezometer, Standard Weir.
EXPERIMENT NO. 14
JUNE 3, 1899.
Diagram Showing Actual Gauge Readings on Piezometers at Standard and Experimental Weirs for Cornell University Experiment No. 14, together with the Periods of Normal Flow and Corrected Heads in Feet used in the Reduction of the Experiments.
Middle Piezometer
Standard Weir
16 Feet in Length
Upper Piezometer
Experimental
Weir
Time, P.M.
Gauge Readings in Centimeters on Up-stream Piezometer, Experimental Weir.

FIG. 8.

In the reduction of Experiments Nos. 20 and 21, the mean readings of the middle piezometer at the upper weir were added to the difference of elevation of the gauge zero and of the mean crest as a basis for computing the flow in the upper channel. The discharge corresponding to the heads so obtained was then taken off the discharge curve computed from Bazin's formula for all the periods in which the observed head was less than 0.6 m. (1.969 ft.). The approximate correction to be applied for velocity of approach to the standard weir was then computed as follows:

Let H = the true head on crest of weir, in feet;

h = the observed head, in feet;

Q_h = discharge over the weir under the head h per lineal foot of crest;

Q = discharge under the head H per lineal foot of crest; and

P = height of weir crest above channel bottom, in feet. Then

velocity of approach $= v = \sqrt{2g(H - h)}$,

also,

$$v = \frac{Q}{P + h} = \frac{Q}{A} \qquad (1)$$

$$\text{and } H_v = (H - h) = \frac{v^2}{2g} = \frac{Q}{A} \times \frac{1}{2g} \qquad (2)$$

Q_h being determined from the discharge curve, an approximate value of v and of the corresponding velocity head was computed and the approximate value of the velocity head so obtained added to the observed head h, which was used in determining Q, v and $(H - h)$ with more precision. Generally, two successive applications of these formulas were found sufficient to determine the velocity head with the desired degree of accuracy. In this way the final corrected head $H = h + H_v$ was obtained. After applying a correction for leakage, percolation and surface evaporation, the corresponding discharges by Bazin's formula have been used in determining the coefficients for the sharp-crested experimental weir (6.53 ft. in length) for heads up to 3.5 ft., as produced by heads not exceeding 2 ft. on the upper weir (16 ft. in length).

The foregoing correction for velocity of approach is merely the addition to the observed heads of $\frac{v^2}{2g}$, as determined for the actual flows of each experiment. Messrs. Fteley and Stearns have experi-

mented on the effect of velocity of approach, especially with reference to that part of it represented by the *vis viva* of the water, and state in their classical paper* that, for the conditions of their experiments, corrections of velocity of approach to be added to the observed heads are best represented by

$$1.45 \text{ to } 1.50 \times \left(\frac{v^2}{2g}\right).$$

The problem of correction for velocity of approach is discussed at length by Hamilton Smith, Jr., M. Am. Soc. C. E., in his "Hydraulics," the conclusion being that, for a weir with full contraction and having an unobstructed channel of considerable length, the correction should be about

$$1.1 \text{ to } 1.25 \times \left(\frac{v^2}{2g}\right).$$

For end contractions suppressed, he adopts the values

$$1.33\left(\frac{v^2}{2g}\right) \text{ and } 1.40\left(\frac{v^2}{2g}\right).$$

In the present case it has seemed preferable to use

$$H_v = \frac{v^2}{2g},$$

although the entire suppression of end contractions might appear to indicate a higher correction for the velocity of approach. This view, however, is based upon other considerations, namely, the actual locations of the piezometers.

Messrs. Fteley and Stearns have pointed out that for standard sharp-crested weirs the head should be measured about 6 ft. back from the crest, but in the present case the heads have been measured much farther back. At the upper 16-ft. weir the heads were measured at the middle piezometer for all experiments, except Nos. 1, 2, 3 and 4, for which they were measured at the upper piezometer. These latter observations have been reduced to the basis of the middle piezometer—which was 10 ft. back from the weir—by methods to be detailed farther on.

At the lower or experimental weirs the heads were measured for all experiments at the upper piezometer 38 ft. above the bulkhead on which the experimental weirs were placed. This location was selected in order to insure the piezometers being well above the long back

* *Transactions*, Am. Soc. C. E., Vol. xii, p. 1.

slopes of Experiments Nos. 2, 3, 14, 15 and 16, or any other similar series which it might appear desirable to make.

Messrs. Fteley and Stearns remark that the only inaccuracy to come from measuring the heads more than 6 ft. back will be that due to surface slope. We will now examine as to the possible effect of this in the present case.

For an observed head of 2.693 ft. on the standard 16-ft. weir

$$\frac{v^2}{2g} = 0.013 \text{ ft.} \qquad (a)$$

and

$$1.33 \left(\frac{v^2}{2g}\right) = 0.017 \text{ ft.} \qquad (b)$$

The difference of 0.004 ft. is far enough within the limit of accuracy to be negligible.

The corresponding observed head on the experimental weir, 6.56 ft. in length, is 4.677 ft., giving

$$\frac{v^2}{2g} = 0.198 \text{ ft.} \qquad (c)$$

also

$$1.33 \left(\frac{v^2}{2g}\right) = 0.270 \text{ ft.} \qquad (d)$$

The difference is 0.072 ft.

Taking the formula $v = C\sqrt{r s}$ in which $r = \frac{A}{P}$ and $s = \frac{h}{l}$ and with l equal to $(38 - 6) = 32$ ft.—the distance between where the piezometer should have been to comply with theoretical conditions for a standard weir and where it was actually set—and computing for h_s under a head of 4.677 ft. on the experimental sharp-crested weir, we find $h_s = 0.061$ ft., which differs from the preceding difference of 0.072 ft. by 0.011 ft. At slightly lower heads this difference disappears so rapidly as to become inappreciable, so far as effect on the coefficients of discharge is concerned.

It was concluded, therefore, that for the conditions of the present case,

$$H_v = \frac{v^2}{2g}$$

gave more nearly the true correction than any other accepted formula.

In his experiments comparing flows over standard weirs with flows over the several experimental sections, Bazin himself did not make

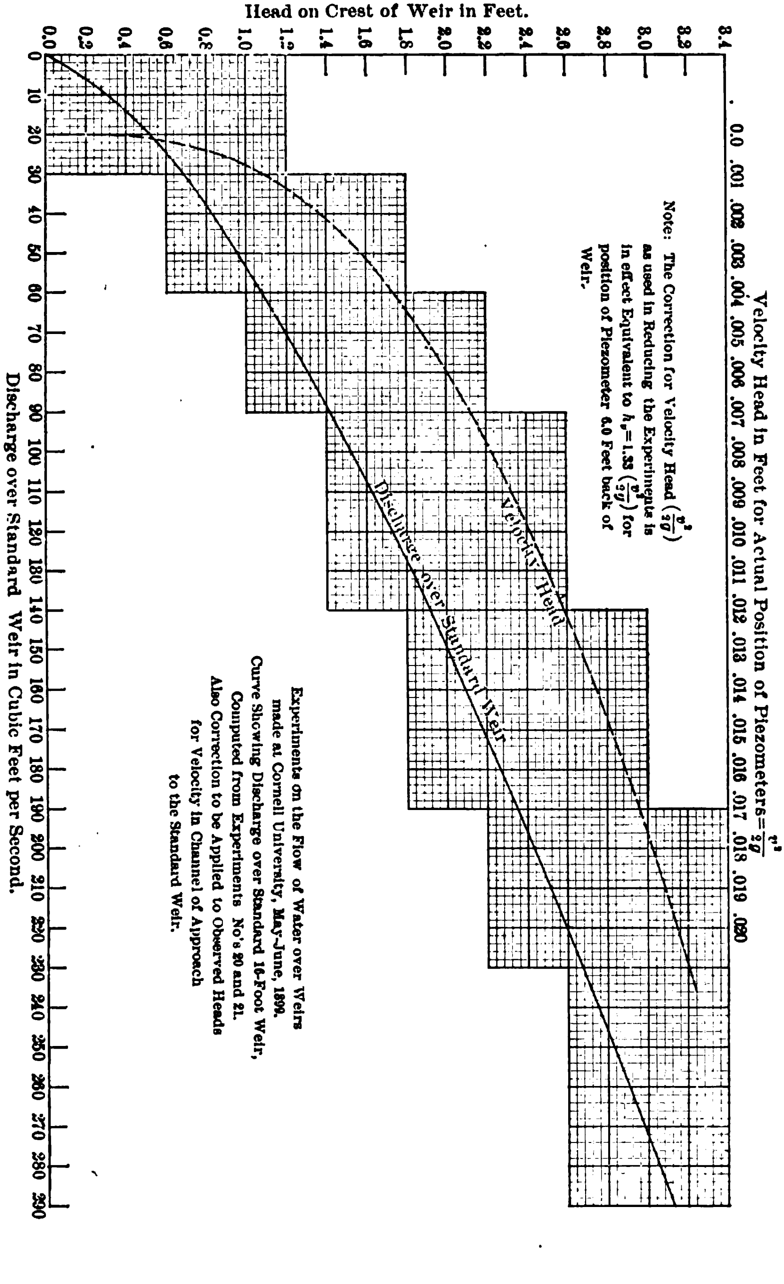

Head on Crest of Weir in Feet.
Velocity Head in Feet for Actual Position of Piezometers $=\frac{v^2}{2g}$
Note: The Correction for Velocity Head $\left(\frac{v^2}{2g}\right)$ as used in Reducing the Experiments is in effect Equivalent to $h_v = 1.33 \left(\frac{v^2}{2g}\right)$ for position of Piezometer 6.0 Feet back of Weir.
Velocity Head
Discharge over Standard Weir
Experiments on the Flow of Water over Weirs made at Cornell University, May-June, 1899.
Curve Showing Discharge over Standard 16-Foot Weir, Computed from Experiments No's 20 and 21.
Also Correction to be Applied to Observed Heads for Velocity in Channel of Approach to the Standard Weir.
Discharge over Standard Weir in Cubic Feet per Second.

FIG 4.

any corrections for velocity of approach. By working on the ratio $\frac{m}{m'}$ the necessity for such corrections was substantially eliminated. On this point, note what he says on page 264.

Final coefficients of discharge for the several weirs have been obtained from the formula

$$C = m\sqrt{2g} = \frac{Q}{LH^{\frac{3}{2}}}$$

in which

Q = total flow over experimental weir, in cubic feet per second;

L = length of crest of experimental weir, in feet; and

H = final corrected head on experimental weir, in feet. Q having been previously found, H was determined from the mean of the readings, for each period of experiment, of a piezometer connected to a horizontal tube placed flush with the bottom of the channel of approach. The correction for velocity of approach was then computed by the formulas just given, using appropriate values of P and h, Q being known from the previous work, as described.

The values of the coefficients m and C, connected by the relation

$$C = m\sqrt{2g},$$

having been determined for the actual heads deduced from the experiments, the values of C so obtained were plotted and a mean curve drawn to best represent the observations. A new series of coefficients advancing by equal increments of H have been read from the curve so obtained, as shown by the tabulations of the experiments, Nos. 1 to 21, on pages 272 to 289, inclusive, and on page 295.

Values of Q, the discharge per lineal foot of crest, have been computed by the formula

$$Q = CH^{\frac{3}{2}}.$$

Having obtained the discharge over the sharp-crested experimental weir for heads up to 3.5 ft., the curve of discharge for the standard weir was extended to the height of 3 ft. by using the coefficients obtained as just described. Fig. 4 shows this curve, as well as the correction curve to be applied to observed heads for velocity in the channel of approach to the standard weir. The additional experiments made on a sharp-crested weir at heads above 3.5 ft. were then reduced in precisely the same manner as before, giving finally a series of coefficients of discharge over a sharp-crested weir with a range of heads from 0.746 ft. to 4.874 ft.

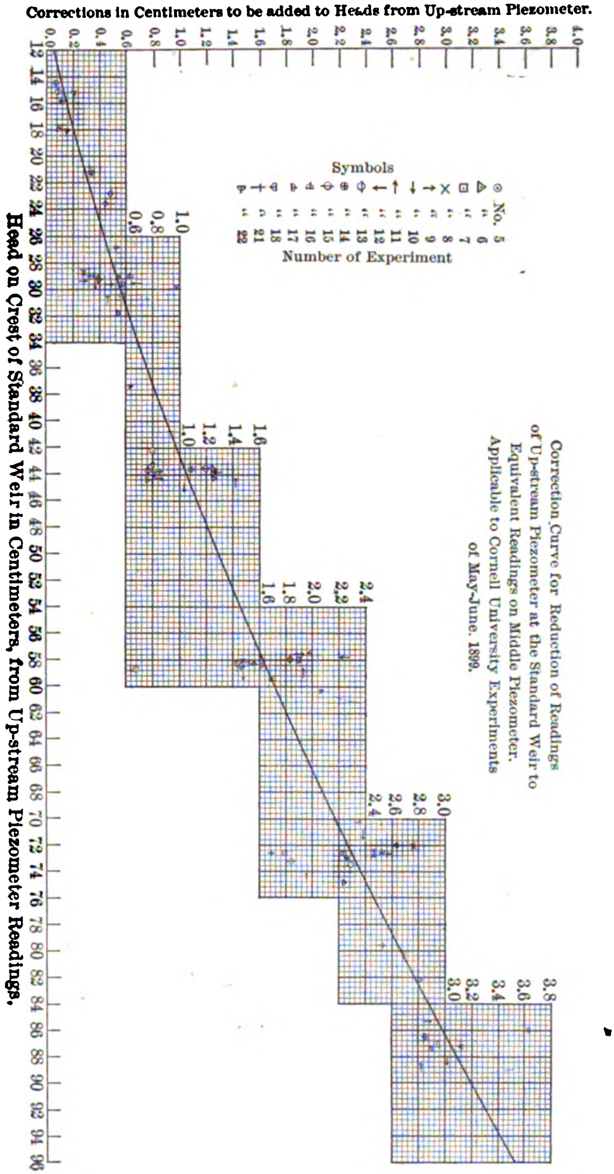

Corrections in Centimeters to be added to Heads from Up-stream Piezometer.
Symbols
No. 5
" 6
" 7
" 8
" 9
" 10
" 11
" 12
" 13
" 14
" 15
" 16
" 17
" 18
" 21
" 22
Number of Experiment
Correction Curve for Reduction of Readings
of Up-stream Piezometer at the Standard Weir to
Equivalent Readings on Middle Piezometer.
Applicable to Cornell University Experiments
of May-June, 1899.
Head on Crest of Standard Weir in Centimeters, from Up-stream Piezometer Readings.

FIG. 5.

The method used in reducing the experiments on weirs having crests of irregular profile is the same as for the sharp-crested weir, with the following exceptions:

In addition to the piezometric observations, direct observations of the head on each weir were taken on gauge boards situated above the standard and experimental weirs, respectively. The agreement between the heads so derived and those deduced from the mean piezometric readings is close in all cases, with the exception of the observed heads on the experimental weir in Experiments Nos. 1, 2, 3 and 4. Without going into the detail, it may be stated that for these four experiments, which were the first made, the heads directly observed on the gauge boards are apparently the more reliable, and they have accordingly been used in reducing these four experiments.

Again, in Experiments Nos. 1, 2, 3 and 4, no observations were taken on the middle piezometer at the standard weir. In order to reduce the observed heads as actually taken on the up-stream piezometer at this weir to equivalent heads on the middle piezometer, a correction has been applied, the value of which was obtained as follows: Plotting the difference between the readings of the upper and middle piezometers as ordinates for all experiments in which readings were taken on both, and using the observed heads on the up-stream piezometer as abscissas, a mean curve has been drawn, from which the correction to be applied to any reading on the up-stream piezometer can be read directly. This curve is shown in Fig. 5. The reason for using the readings of the middle, in preference to those taken on the up-stream piezometer at the standard weir, is that the former agree more closely, on the whole, with the readings of the gauge board; also, the middle piezometer, which is only 10 ft. distant from the bulkhead, on which the standard 16-ft. weir was located, is more nearly at the proper distance from the weir. Moreover, the up-stream piezometer was situated so far back from the standard weir as to be evidently disturbed somewhat by the entrance velocity of the water in the leading channel.

With the exception of Experiments Nos. 18, 19, 20 and 21, the readings at the experimental weir were taken from a piezometer placed horizontally across the channel of approach at a height of about 8 ins. above the bottom. In order to reduce the readings from this piezometer to the equivalent readings from the piezometer placed

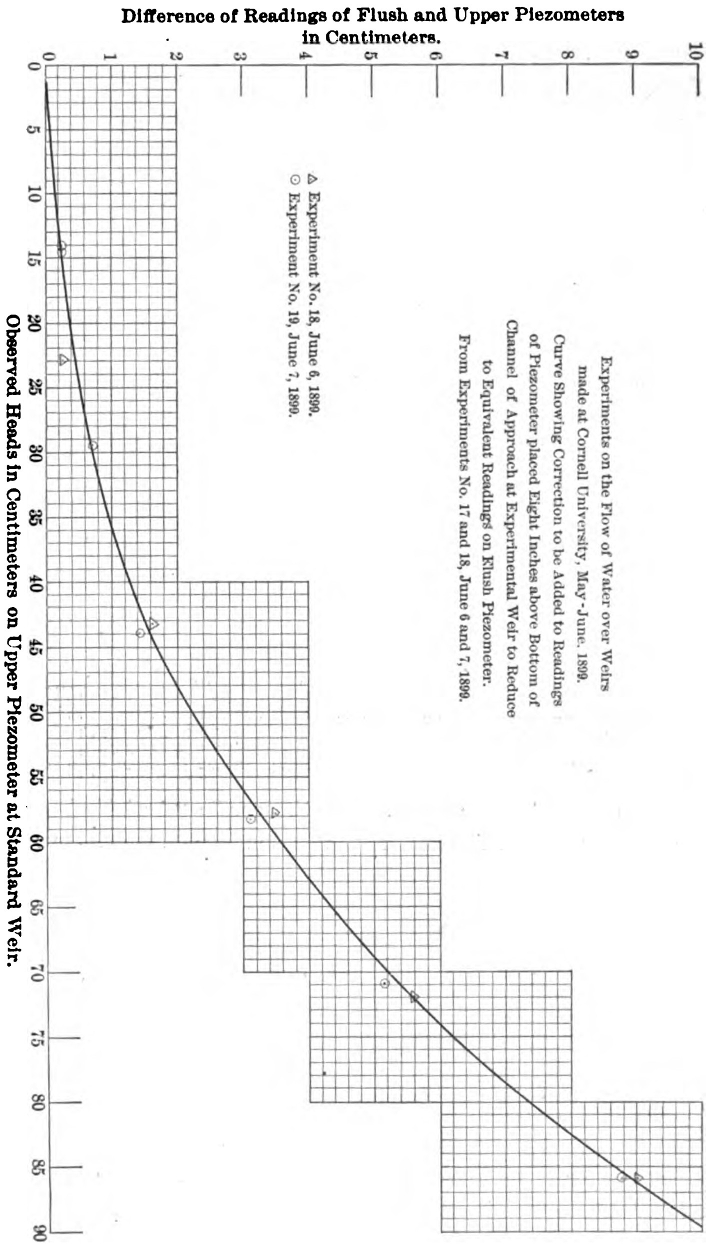

Difference of Readings of Flush and Upper Piezometers in Centimeters.
0 1 2 3 4 5 6 7 8 9 10
Experiments on the Flow of Water over Weirs made at Cornell University, May-June. 1899.
Curve Showing Correction to be Added to Readings of Piezometer placed Eight Inches above Bottom of Channel of Approach at Experimental Weir to Reduce to Equivalent Readings on Flush Piezometer.
From Experiments No. 17 and 18, June 6 and 7, 1899.
△ Experiment No. 18, June 6, 1899.
⊙ Experiment No. 19, June 7, 1899.
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90
Observed Heads in Centimeters on Upper Piezometer at Standard Weir.

FIG. 6.

flush with the bottom of the channel, a correction curve has been deduced in the following manner: In Experiments Nos. 18 and 19 observations were taken, both from the flush piezometer and from one situated 8 ins. above the bottom. Plotting the differences between the readings of these two piezometers corresponding to given heads on the standard weir as ordinates, and using the observed heads on the standard as abscissas, a mean curve has been drawn, from which a correction to be applied in any case can be read directly. This curve is shown Fig. 6.

In regard to the use of this correction curve, it may be pointed out that the error resulting from the use of a piezometer placed otherwise than flush with the bottom or side of the channel is a function of the velocities in the channel of approach. Inasmuch as the discharging capacities of weirs of different sections vary greatly under the same heads, the velocity of approach at any given head will depend both

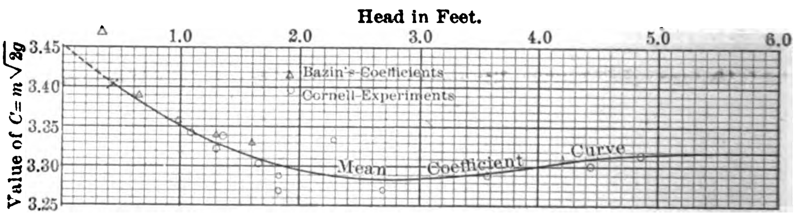


FIG. 7.

upon the height P of the weir and on the coefficient of discharge for the particular head and section of weir considered. Unfortunately, the data obtained were insufficient to enable the effect of these elements to be taken into consideration separately. Hence, the corrections obtained from the mean curve of Fig. 6 must be considered as approximate only. Some of the deviation of the experimental coefficients from the mean coefficient curves may undoubtedly be attributed to the uncertainty as to just the proper value of this correction.

Pages 272 to 289 show the coefficient curves finally fixed upon by the foregoing discussion for the Cornell University Experiments Nos. 1 to 19, inclusive, and also the tabulations of the results. The coefficient curve for Experiments Nos. 20 and 21 on the standard sharp-crested weir is shown in Fig. 7. These curves are so self-explanatory as to render extended description unnecessary.

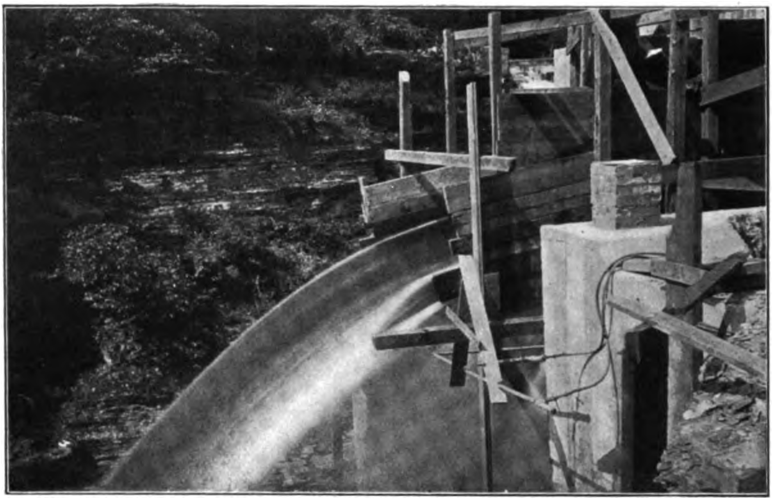


FIG. 1.—LOWER END OF CHANNEL. CORNELL UNIVERSITY HYDRAULIC LABORATORY.

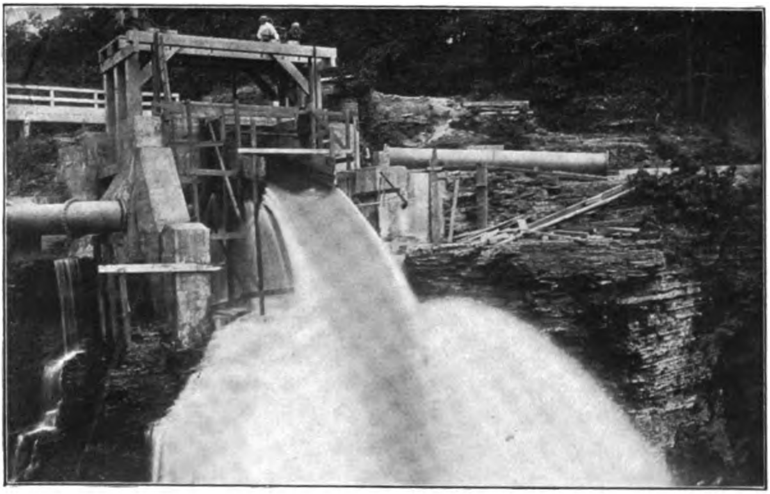


FIG. 2.—LOWER END OF CHANNEL. CORNELL UNIVERSITY HYDRAULIC LABORATORY.

In the tabulations Column (1) shows the heads in feet, as read from the coefficient curves, Column (4) giving the discharge per lineal foot of crest in cubic feet per second. Columns (2) and (3) give the values of the coefficients m and C.

The photographs on Plate XXII show the lower end of the lower channel, as it appeared on the afternoon of June 3d, 1899, while experiments on the Rexford Flats section were in progress.

It is not the writer's intention to review extensively the results of the Cornell University Experiments at this time, any farther than to point out that they were, in the fullest sense, practical experiments.

In Experiments Nos. 7 and 8 an attempt was made to gain some idea of the effect of a rough surface on dams. In Experiment No. 7 the weir was of the usual form, with the crest constructed of planed matched pine, as already described, while in Experiment No. 8 the up-stream face of the crest was covered with ¼-in. mesh, galvanized wire screen. A comparison of these two experiments is very instructive. The upper limiting head of No. 7 was 4.996 ft. and of Experiment No. 8, 5.011 ft. For 5 ft. head, as determined from the curves, we have for Experiment No. 7, a discharge of 40.98 cu. ft. per second per lineal foot of crest, while for Experiment No. 8, 5 ft. head gives a discharge of 40.74 cu. ft. per second per lineal foot of crest. Similar comparisons at other heads show the effect of the wire screen to have been but slight. The result of this experiment was such as to lead to the conclusion that very little difference would be experienced in the flow over a dam after the first few months, during which time the planking, under the smoothing effect of the silt in flowing water, etc., may be expected to come substantially to the hydraulic condition of planed boards. Accordingly, as the time for completing the work was limited, no further determinations were made on this line.

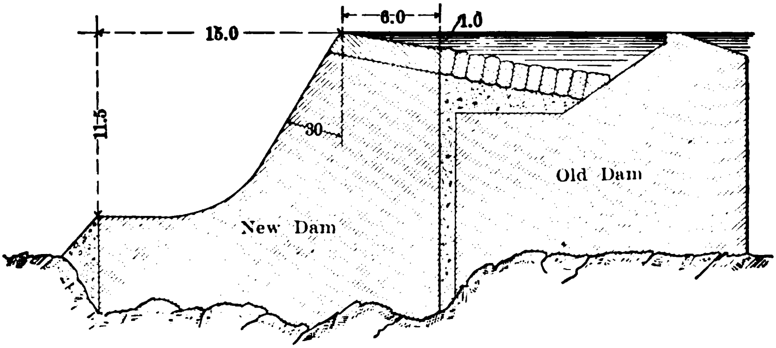
It is recognized, however, that but for the limitation of time under which the experiments were carried out, it would have been very desirable to have experimented somewhat farther on a number of additional forms of weirs, and it is to be hoped, in view of the vast practical importance of an accurate knowledge of flow over dams, that the Cornell University authorities will carry these experiments considerably farther, keeping especially well within the limits of actual practice in dam construction.

Application of Data to Cases in Practice.

We may now consider the application of the foregoing data to some of the dams at several of the gauging stations previously referred to.

Seneca River at Baldwinsville.—At Baldwinsville Station on Seneca River, there is a substantial masonry dam, as shown by Fig. 8. It was built in 1895, taking the place of an old crib dam located just above, as shown in the illustration. The crest is 423 ft. long, and is very nearly level. The catchment area of Seneca River, at Baldwinsville, is 3 103 sq. miles.

For most of the year flash boards, 1 ft. in height, are used on a portion of the crest. The flow over these has been computed by



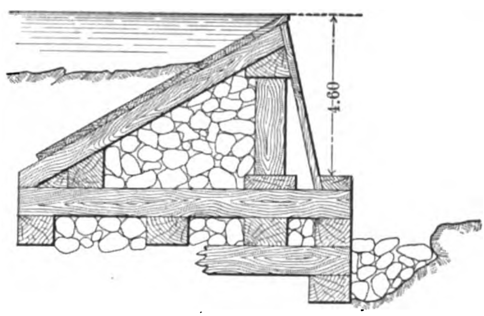
CROSS-SECTION OF DAM ON SENECA RIVER AT BALDWINSVILLE.

Fig. 8.

Francis' formula. The situation is somewhat complicated by the presence of the old dam. Taking into account the water cushion between the two crests, it was considered that Bazin's Series No. 115 would fairly apply, and, accordingly, the discharge curve was computed on this basis. The conditions here are so unusual that a special determination should be made as a check on the foregoing assumption. This was not done, during the Cornell University Experiments, for lack of time.

Oswego River at Fulton.—The catchment area above this dam is 4 916 sq. miles. There are extensive manufacturing establishments at the ends. The dam is a substantial masonry construction with a nearly vertical front, and with a back slope of 1 to 8. The

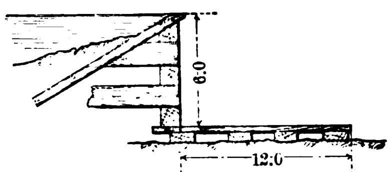
crest is slightly rounded. Bazin's section making the nearest approach to this is Series No. 117, which was recognized as being only an approximation. Cornell University Experiment No. 3 has furnished the data for working out a new discharge curve applying more nearly to the conditions of this dam than the original. It is believed that the revised curve gives the true discharge within a small percentage. At a depth of 2 ft. on the crest, the discharge, by the revised curve, is 9.8% less than by the original curve.



CROSS-SECTION OF DAM ON CHITTENANGO CREEK AT BRIDGEPORT.

FIG. 9.

Chittenango Creek at Bridgeport. — The cross-section of this dam is shown by Fig. 9. The catchment area above the point of gauging is 307 sq. miles, while the crest is 259.2 ft. in length. At the ends there are platforms over the bulkheads, and about 2.5 ft. above the main crest. The flow over these platforms when the water rises to their height, was originally computed by Bazin's Series No. 113, while Series No. 130 was applied to the main crest, shown by Fig. 9. The revised discharge curve for this dam is based upon Cornell University Experiments Nos. 1 and 10. The computed discharge, as per the revised curve, is—at a depth of 2 ft. on the crest—11.9% less than by the original curve.



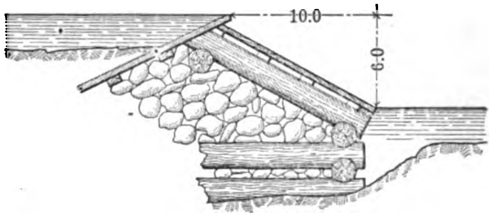
CROSS-SECTION OF DAM ON ONEIDA CREEK AT KENWOOD.

FIG. 10.

Oneida Creek at Kenwood.— The catchment area above this dam is 59 sq. miles. The crest is level, and is 79.4 ft. in length (see Fig. 10). The cross-section corresponds closely to Bazin's Series Nos. 130 or 135. The final discharge curve has been worked up from Cornell University Experiment No. 2.

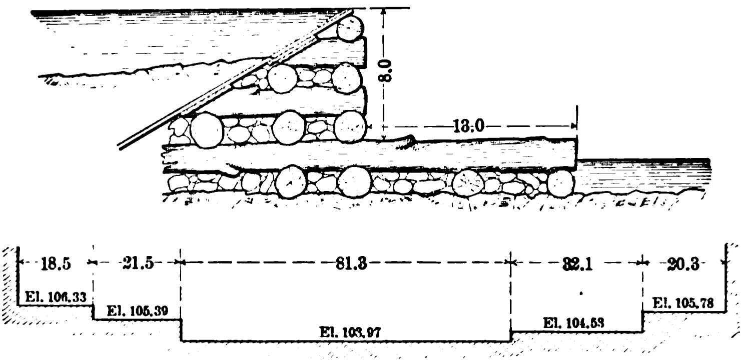
West Branch of Fish Creek at McConnellsville.—The catchment area above this dam is 187 sq. miles. The crest was originally quite irregular longitudinally, but was brought to a nearly uniform level by

spiking on strips of plank, which extended down the back or up-stream face. The length is 175.7 ft. (see Fig. 11). Bazin's Series No. 170 conforms in its general form closely to the cross-section, except that the projection of the planking of the back face over the front, forms an air space which has a disturbing effect on low flows. For minutely accurate results, on such a profile, special determinations should be made. Cornell University Experiment No. 7 has been used in preparing the final discharge curve.



CROSS-SECTION OF DAM ON WEST BRANCH OF FISH CREEK AT McCONNELLSVILLE.
FIG. 11.

East Branch of Fish Creek near Point Rock.—The catchment area above this dam is 104 sq. miles. The crest is 173.7 ft. in length, and is at several different heights, as shown in Fig. 12. Bazin's Series No. 130 applies closely. The final discharge curve is based upon Cornell University Experiment No. 1.

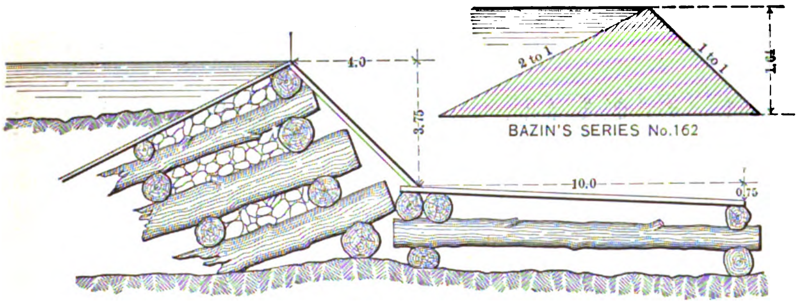


CROSS-SECTION AND PROFILE OF DAM ON EAST BRANCH OF FISH CREEK NEAR POINT ROCK.
FIG. 12.

Mohawk River at Ridge Mills.—The catchment area is 153 sq. miles. The crest is 122.7 ft. long, and is at three different elevations. As shown by Fig. 13, the experimental and actual sectional profiles agree closely.

Bazin's Series No. 162 is of such a form that the discharge is nearly

uniform at all heads. The syphon action of the sloping front face begins at low heads and continues to act with indefinitely increasing heads. There is no point where marked changes in regimen occur, as with depressed and adhering nappes. The coefficient of Cornell University Experiment No. 6 agrees closely with Bazin's No. 162. A discharge curve, based upon Bazin's No. 162, varies at 2 ft. depth on crest, only 1.5% from the discharge curve by Cornell University Experiment No. 6. Crests of this general form are especially applicable wherever accurate records of flow are required. For reasons given by Bazin, in the preceding abstracted matter, crests of this general form, but with flat upper surface, should be avoided. On this point see page 268 preceding.



CROSS-SECTION OF DAM ON MOHAWK RIVER AT RIDGE MILLS,
IN COMPARISON WITH BAZIN'S SERIES No. 162.

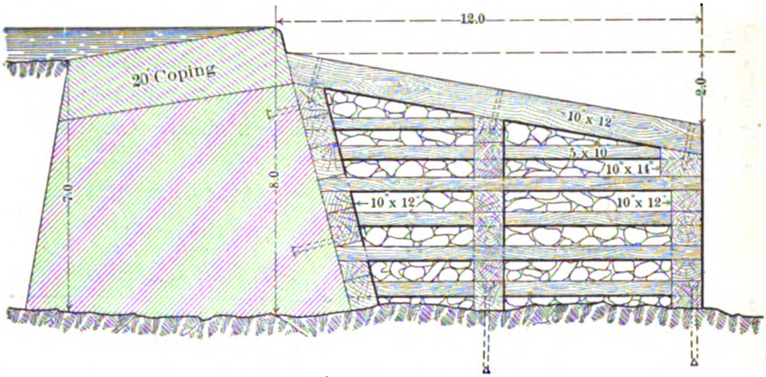
FIG. 13.

Mohawk River at Little Falls.—The catchment area at this place is 1 306 sq. miles. The dam is of well-built masonry, curved in plan, with a crest 181.7 ft. in length. There are two sections, with cross-sections corresponding to Cornell University Experiments Nos. 16 and 17, except that the dam has batters on the front face for the two sections, 1 : 6 and 1 : 4, respectively. Inasmuch as there was full admission of air in the experiments, this fact would not affect the results, although the projection of the front face of the dam, itself, due to the batter may introduce disturbing elements in the flows, which would modify somewhat the results of Cornell University Experiments Nos. 16 and 17. The vertical front was used in the experiments in order to expedite the work.

The discharge curve worked out originally for this dam, was based upon Bazin's Series Nos. 117 and 135. A new curve, derived from

Cornell University Experiments Nos. 16 and 17, gives, for heads of 2 ft., 8.8% less flow than the original.

Mohawk River at Rexford Flats.—The catchment area at this place is 3 385 sq. miles. The dam is a substantial masonry construction with a timber apron, as shown by Fig. 14. The crest is 675 ft. long. The filling at the back of the dam makes, in effect, a long flat crest on the

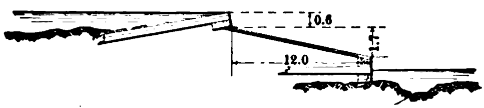


DAM ON MOHAWK RIVER AT REXFORD FLATS.

FIG. 14.

up-stream side. In the original discharge curve, Bazin's Series Nos. 117 and 141, were taken as applying best. This dam was experimented upon at Cornell University, the new discharge curve resulting therefrom agreeing very closely with the original curve. The variation in the two curves at 2 ft. depth on the crest is only about 1 per cent.

Oriskany Creek at Oriskany.—The catchment area here is 144 sq. miles. The crest is 214 ft. in length, and is at three different elevations (see Fig. 15). For the original discharge curve, a mean of the coefficients of Bazin's Series Nos. 117 and 141 were considered to apply best. A revised discharge curve, based upon Cornell University Experiment No. 14, has been worked out. At a depth of 2 ft. the new curve increases the discharge 1.7 per cent.



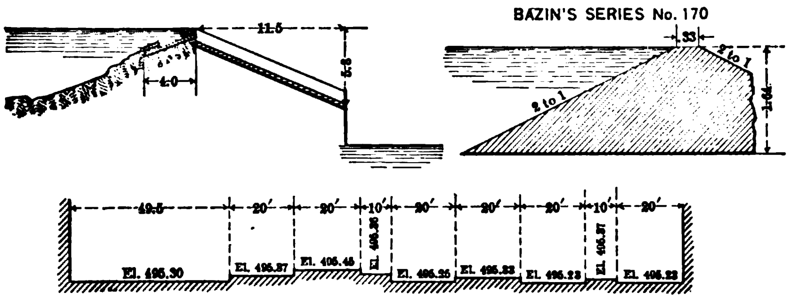
CROSS-SECTION OF DAM ON ORISKANY CREEK AT ORISKANY.

FIG. 15.

Oriskany Creek at Coleman.—This station is a little more than a mile above the dam at Oriskany, just described. The catchment area is

141 sq. miles, or 3 sq. miles (2.2%) less. Fig. 16 shows the cross-section in comparison with Bazin's Series No. 170, as well as the irregularities of the crest longitudinally. The remarks previously made as to flow, on Bazin's Series No. 162, apply to No. 170 and other sections of similar form. The disturbing effect of the departure from the theoretical form is unknown in this case, the same as for the dam on Mohawk River at Ridge Mills.

The object of establishing two stations on Oriskany Creek was to determine whether on dams of different forms, but with nearly the same catchment areas, the flows could be gauged closely enough to give fairly comparable figures. The following tabulation gives the



CROSS-SECTION AND LONGITUDINAL PROFILE OF DAM ON ORISKANY CREEK AT COLEMAN, IN COMPARISON WITH BAZIN'S SERIES No. 170.

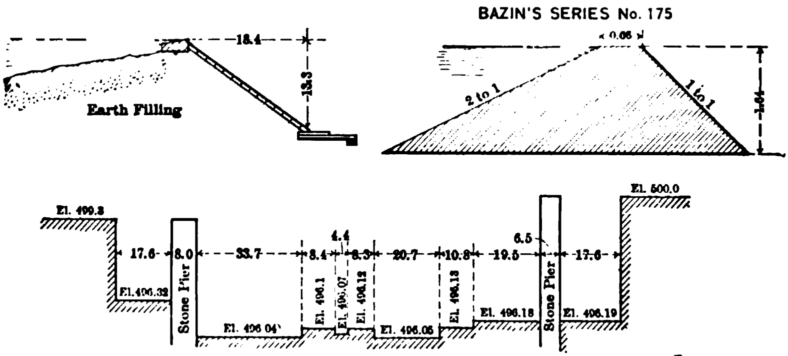
FIG. 16.

flows at Oriskany and Coleman, in cubic feet per second, for the months from November, 1898, to February, 1899, inclusive. During the frozen months of this period, the ice was kept clear for several feet back from the crest of each dam. The results show fair agreement, and indicate, that, even when one of the cases is complicated, as at Coleman, by discharge through several water wheels, comparable results may still be gained.

Month.	Discharge at Oriskany.	Discharge at Coleman.
November	327	306
December	327	335
January	295	297
February	291	283

Saquoit Creek at New York Mills.—The catchment area here is 52 sq. miles. The crest is as shown by Fig. 17. Bazin's Series No. 175 is taken as applying best to the main section. For the flash boards at the end sections, Francis' formula has been used.

West Canada Creek at Middleville.—The catchment area above this dam is 519 sq. miles. The crest is 330.5 ft. in length, and is leveled up, as described for the dam at McConnellsville (see Fig. 18). The original discharge curve was based upon Bazin's Series No. 170. A new curve based on Cornell University Experiment No. 15 (Rexford Flats section with rounded corner) gives 23% less discharge at 2 ft. depth than the original. The writer considers that this section should be specially determined, for accurate results, and the foregoing



CROSS-SECTION AND LONGITUDINAL PROFILE OF DAM ON SAQUOIT CREEK AT NEW YORK MILLS, IN COMPARISON WITH BAZIN'S SERIES No. 175.

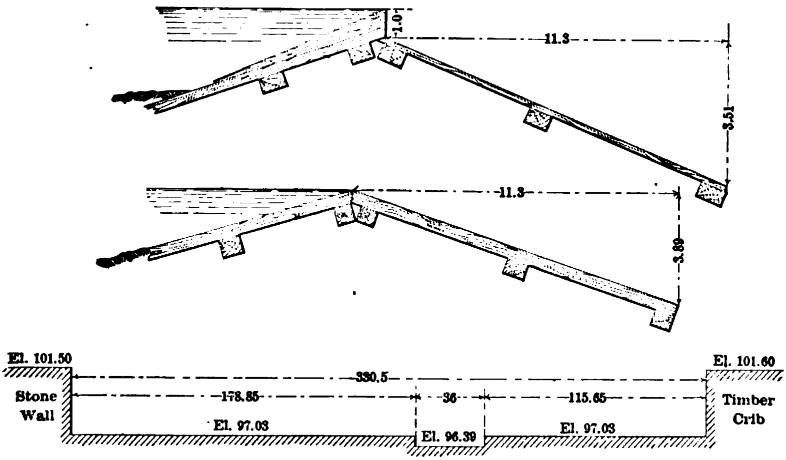
FIG. 17.

discrepancy is cited merely to show how useless it is, if accurate results are required, to apply the nearest form at hand. The whole study shows that, frequently, what appears at first sight to be relatively unimportant produces very marked changes in discharge.

Cayadutta Creek near Johnstown.—The tributary catchment area is 40 sq. miles. The main section, Fig. 19, is quite different from any of the dams thus far considered. Bazin's Series No. 130, was taken as being nearer than any other, while Series No. 115 was applied to the bulkheads at the ends. Where a high degree of accuracy is required for gaugings over a nondescript section of this sort, special experiments must be made.

Schoharie Creek at Fort Hunter.—The catchment area above this dam is 947 sq. miles. As shown by Fig. 20, it has a sectional profile similar to those of the dams on Oriskany and West Canada Creeks. The original discharge curve was based upon Bazin's Series Nos. 117 and 141. A re-computation, using Cornell University Experiment No. 14, gives substantially the same curve.

The foregoing account of several applications of the new views as to flow over dams has been made as concise as possible in order not to lengthen this paper unnecessarily. Matters of interest relating to the leakage of dams and flumes, methods of computing discharge through nondescript water wheels, and many other questions, are pur-



CROSS SECTIONS AND PROFILE OF DAM ON WEST CANADA CREEK
AT MIDDLEVILLE.

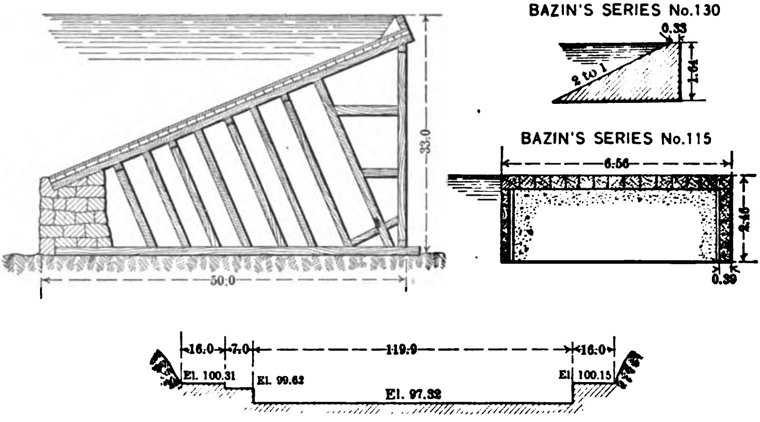
FIG. 18.

posely left untouched, in order to discuss more thoroughly the main question of how to compute the flow over dams. The Schoharie Creek dam may be especially mentioned as one with considerable leakage, and which is used here for illustrative purposes only.

In passing, it may be remarked that one result of the Cornell University Experiments was to show that the discharges, as per Bazin's Series Nos. 130 and 135, were much too high, especially at the considerable heads occurring at several of these gauging stations. The reasons for this are found, apparently, in the high discharges accompanying the depressed and adherent nappes, which occur at the low

heads Bazin experimented with. This, Bazin himself has pointed out, in the matter abstracted from his last paper, on a previous page. The great difference, however, is only fully realized when we carry out comparable experiments to the heads used at Cornell University.

The different methods of experimentation may also be taken into account. Bazin usually began with a low head, gradually increasing to the higher; whereas, at Cornell University, in all the experiments, the high heads were run first, and were gradually reduced. In both cases, the established regimen of flow, whatever it may have been, was continued longer than would have occurred under the contrary condition, the coefficients for the two states lapping by one another.* The



DAM ON CAYADUTTA CREEK NEAR JOHNSTOWN, ETC.

FIG. 19.

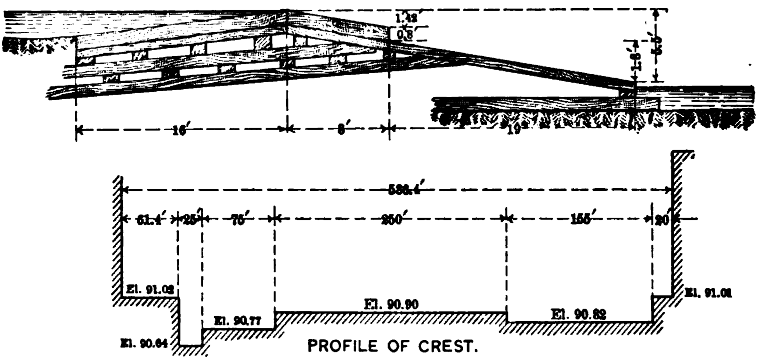
conclusion under this head is, therefore, that for a rising stream the discharge at or near the critical point of change, may be appreciably different from the discharge for a falling stream at about the same point.

By way of illustrating the method of computation used, we may discuss the computation for Schoharie Creek dam. Table No. 4 shows how the data for the discharge curve for this dam have been arrived at, the coefficients used therein being derived from Cornell University Experiment No. 14. To begin with, zero of the crest gauge is at elevation 90.68 ft. The crest itself divides into a series of sec-

* As to just the condition of the nappes for Bazin's Series Nos. 130 and 135, and several similar sections, see the tabulations of Bazin's Series, on preceding pages.

tions, with elevations as shown on the longitudinal profile of Fig. 20, and which are designated in the table, by the letters *A*, *B*, *C*, *D*, *E*, and *F*.

The method of procedure for the computation of points for the discharge curve is as follows: The average elevation of each section of dam—*A*, *B*, *C*, etc.—having been computed with reference to zero of the crest gauge, the depth of water flowing over each section, corresponding to a series of readings on the gauge, was deduced and tabulated, as shown. Thus, for section *A*, we have, in Column (4), head on section in feet, and so on for the other sections. Column (4) also includes the discharge per lineal foot of crest, for heads ranging from 0.2 ft. up to 8.0 ft., together with the total flow per section, for the same heads. These computations are made on the basis of



PROFILE OF CREST.

SECTION AND PROFILE OF SCHOHARIE CREEK DAM.

FIG. 20.

no end-contraction at the ends of the sections. The summation at the foot of the table represents the total flow over the dam, and ranges from 26 cu. ft. per second, for a head of 0.2 ft. to 41 182 cu. ft. per second, for a head of 8.0 ft. The discharge curve is constructed by plotting the final footings, with the heads on the crest in feet as ordinates, and discharges in cubic feet per second, as abscissas.

The foregoing general method has been applied, with necessary variations to fit each special case, to all the gauging stations herein referred to.

On examining the values of $C = m\sqrt{2g}$ in the coefficient tables given herewith, the great range in discharge, not only for different

TABLE No. 4.—SHOWING METHOD OF COMPUTING DISCHARGE CURVE FOR DAM ON SCHOHARIE CREEK AT FORT HUNTER.

Discharge Coefficients are from Cornell University Experiment No. 14.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Section.	Length, in feet.	Elevation, in feet.	Head above gauge zero.	0.2	0.6	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0
			Head on Section *A*, feet.....	......	0.27	0.67	1.67	2.67	3.67	4.67	5.67	6.67	7.67
A........	20	91.01	Discharge per foot of crest.	......	0.37	1.62	6.82	14.13	22.73	32.62	43.62	55.80	68.30
			Total flow over section........	......	7.40	32.40	136.00	283.00	455.00	652.00	872.00	1 116.00	1 366.00
			Head on Section *B*, feet.....	0.06	0.46	0.86	1.86	2.86	3.86	4.86	5.86	6.86	7.86
B........	155	90.82	Discharge per foot of crest.	0.06	0.90	2.40	8.20	15.70	24.65	34.65	45.90	58.00	70.85
			Total flow over section......	9.30	139.50	372.00	1 271.00	2 434.00	3 821.00	5 371.00	7 115.00	8 990.00	10 982.00
			Head on Section *C*, feet.....	......	0.38	0.78	1.78	2.78	3.78	4.78	5.78	6.78	7.78
C........	250	90.90	Discharge per foot of crest.	......	0.70	2.15	7.60	15.10	23.90	33.95	45.00	57.05	69.80
			Total flow over section......	......	175.00	538.00	1 900.00	3 775.00	5 975.00	8 488.00	11 250.00	14 263.00	17 450.00
			Head on Section *D*, feet.....	0.11	0.51	0.91	1.91	2.91	3.91	4.91	5.91	6.91	7.91
D........	75	90.77	Discharge per foot of crest.	0.10	1.10	2.62	8.46	16.20	25.05	35.20	46.50	58.50	71.50
			Total flow over section......	7.50	83.00	196.00	634.00	1 215.00	1 879.00	2 640.00	3 488.00	4 388.00	5 363.00
			Head on Section *E*, feet.....	0.24	0.64	1.04	2.04	3.04	4.04	5.04	6.04	7.04	8.04
E........	25	90.64	Discharge per foot of crest.	0.36	1.77	3.28	9.40	17.20	26.40	36.60	48.05	60.30	73.30
			Total flow over section......	9.00	44.00	82.00	235.00	430.00	660.00	915.00	1 201.00	1 508.00	1 833.00
			Head on Section *F*, feet.....	......	0.26	0.66	1.66	2.66	3.66	4.66	5.66	6.66	7.66
F........	61.4	91.02	Discharge per foot of crest.	......	0.36	1.60	6.80	14.10	22.70	32.58	43.58	55.75	68.20
			Total flow over section......	......	22.00	98.00	417.00	865.00	1 394.00	2 000.00	2 676.00	3 423.00	4 187.00
Total flow over dam, in cubic feet per second...........				26.00	471.00	1 318.00	4 593.00	9 002.00	14 184.00	20 066.00	26 602.00	33 688.00	41 182.00

Elevation of zero of crest gauge = 90.68 ft.

forms of dams, but for varying heads, becomes apparent. These tables indicate that the making of accurate gaugings over dams demands considerable skill in the application of the available information. For minutely accurate results, special experimentation is, in many cases, indispensable.

As regards experiments on dams, the Cornell University Hydraulic Laboratory can hardly be improved upon, and the University authorities deserve the sincere thanks of every hydraulician, for furnishing an equipment of this character. It is to be hoped that data of flow over dams may be greatly extended there in the next few years.

In concluding the paper, the writer may remark that the studies herein discussed were, in reality, only a side issue of the entire investigation of the water-supply problems carried out for the United States Board of Engineers on Deep Waterways. For whatever deficiencies may appear, the writer hopes he may be pardoned, because of the time limit set by the Board, which was, the completion of everything within one year. This condition compelled a strictly business-like administration and the omission of much purely scientific detail which, with more time available, it would have been very pleasant to pursue somewhat farther. It was necessary, indeed, under the requirements of the Board, to be first of all a business man—driving the matter in hand along rapidly to a final conclusion—and only indulging in pure scientific work so far as this did not conflict with definite progress from day to day.

As regards the Cornell University authorities, the conditions are different, and they will without doubt ultimately supply the engineering profession with far more extended knowledge of the flow of water over dams, especially at high heads, than is now possessed.

The total cost of this set of experiments, including materials, common labor, carpenters, engineering assistants, draughtsmen, stenographer and time of writer did not exceed $1 800. This figure does not include either Professor Williams' time or cost of gauges, which were paid for by the University as part of the permanent equipment, but it includes all payments on account of these experiments by the United States Board of Engineers on Deep Waterways.

The writer's thanks are due to Professor E. A. Fuertes, Director of Cornell University College of Civil Engineering, for many courtesies received during the progress of the study.

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PAPERS AND DISCUSSIONS.

THE PRACTICAL COLUMN UNDER CENTRAL OR ECCENTRIC LOADS.

By J. M. MONCRIEFF, M. Am. Soc. C. E.

TO BE PRESENTED MAY 2D, 1900.

This subject has received the attention of many investigators, and, in consequence, numerous formulas have been laid before the engineering profession, with the object of providing a means of predicting or estimating the probable strength of columns as affected by their proportions. Nevertheless, it appears to the writer that no apology is needed for a fresh attempt in this direction.

In developing a theory of column resistance and its resulting formulas some of the more important points requiring to be kept in view are the following:

1. The theory should be based on correct principles and the formulas should be of correct form, without introducing refinements which have but little practical value or influence on the results.
2. The theory and formulas should have a wide range of application, to cover the conditions met in engineering practice,

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and should not be limited to the case of columns under presumably central loads.

3. The reasoning and the formulas should be sufficiently simple to be understood by other than expert mathematicians, and should be suitable for use in every-day engineering practice.
4. The theory must show agreement with, and be a reasonable explanation of, the results of practical experiment.
5. The formulas should in practice be applicable generally to different materials, by simply introducing the values of the ordinary physical constants of strength and stiffness.
6. Empirical factors should be reduced to a minimum.

The simple theory and formulas which are set forth in this paper show a fair compliance with the foregoing requirements, and it is hoped that they may assist the practical engineer to a better understanding of the important subject of column strength.

The underlying principles upon which the reasoning is based are, (1) that a perfectly centered column of perfect material and straightness is an ideal conception seldom or never realized in practice, and (2) that the various disturbing influences preventing the realization are practically all capable, as regards their ultimate effect, of being represented by an equivalent eccentricity of loading.

Any theory based on these principles ought to be identical in its results with the theory of the ideal, perfectly centered column of perfect material and straightness, when the factor representing eccentricity is reduced to zero.

The assumption of the principle of equivalent eccentricity receives practical justification in the records of the experiments of James Christie, M. Am. Soc. C. E.; the late Charles A. Marshall, M. Am. Soc. C. E.; M. Considère, and Professor Bauschinger, each of whom found that the physical axis of resistance in a column did not necessarily coincide with the geometrical axis, and in fact very frequently did not. Each of these experimenters made tests in which he practically felt for the physical axis in order to obtain a higher strength for the column under trial, and found that it was quite possible for a column to show higher strength when apparently loaded eccentrically, as compared with the strength when apparently loaded exactly over

the geometrical axis. Mr. Christie's and Mr. Marshall's principal tests were both, however, centered over the geometrical axis, and their attempts to feel for the physical axis were supplementary.

After the development of the theory and its resulting formulas, the first difficulty in its application to the case of columns under apparently central loading is the value to be assigned to the equivalent eccentricity, and a careful study of the records of nearly all the more important tests of columns was therefore undertaken, with the view of arriving at some idea of what that value should be.

It became at once apparent, from a comparison of the tests by different experimenters, that isolated tests or a set of tests covering only a small range in proportions, as measured by the ratio of length to radius of gyration, or having only a scanty number of tests at each ratio $\frac{l}{r}$, could in themselves afford no reliable basis for use in practical work, and the writer has no hesitation in saying that any general conclusions or formulas derived from such conditions are absolutely misleading.

It also became most clearly evident that any conclusions deduced from experiments must make full allowance for the possible or probable history of the material of the column during its manufacture and during its preparation for the testing machine.

Many of the causes of the divergence of practical experiments from the theoretic ideals have been incidentally alluded to in able papers and discussions on the subject of column resistance, but, very frequently, too much emphasis has been laid upon the variable nature of the material—excepting perhaps in the case of timber—and, on the other hand, too little has been credited to the probable history of the material, and to the influence of apparently insignificant initial curvature in the specimens, or small errors in setting in the testing machine.

The divergence alluded to may, with every probability of truth, be partly credited, in the case of wrought iron and steel, to the effects of the inevitable cold-straightening to which every bar, plate, or shape, turned out of the rolling mill, must be subjected before being fit for use in ordinary construction, and to a still greater degree before being put into a testing machine as a properly prepared specimen.

It cannot be too clearly realized that the material used in every-day construction is in anything but an ideal condition as regards freedom

from internal stresses, and as regards uniformity of elastic resistance in its detailed sections.

This is quite a different thing from assuming that material of similar history and of the same class varies very widely in its compressive strength, or in the value of the modulus of elasticity.

Striking instances of the influence of history have been given by *Sir* Benjamin Baker, Hon. M. Am. Soc. C. E., in the case of experiments which he carried out on solid, mild-steel columns, 30 diameters in length, showing that the resistance varied according to previous treatment, as follows:*

	Tons per square inch.
"Annealed	14.5
Previously stretched, 10 per cent	12.6
" compressed, 8 "	22 1
" " 9 "	28.9
Straightened cold	11.8"

There are also a number of references to the influence of history in Mr. James Christie's† papers on the strength of iron and steel.

The effect of cold-straightening is, of course, to locally strain the material beyond the limit of elasticity, and, without this overstrain, the bar or plate could not be straightened. The result is that at certain points the modulus of elasticity, or, in other words, the stiffness of the fibers overstrained in tension, is lowered very greatly as regards resistance to compressive stress, and the fibers overstrained in compression are affected similarly as regards their resistance to tensile stress. In addition, permanent internal stresses, both tensile and compressive, are set up in the material, and these are neither imaginary nor insignificant.‡

A direct consequence of this interference with natural conditions is that the "physical" axis, or the axis passing through the center of resistance of every section of the column, will not be coincident with the geometrical axis, and in forming a mental conception of the physical axis under these artificial conditions, we are driven to the conclusion that in practical work it can rarely, if ever, be a straight line.

If these deductions be extended to the case of short test-specimens under compression, how is it to be expected that accurate determina-

* *Minutes of Proceedings*, Institution of Civil Engineers, Vol. xcii, p. 44.
† *Transactions*, Am. Soc. C. E., Vol. xiii.
‡ *Minutes of Proceedings*, Institution of Civil Engineers, Vol. xcii, p. 44.

tions of the natural elastic compressive strength can be derived from specimens cut from a portion of the material which may previously have been subjected to cold-straightening. Other portions of the same piece may not have been in the straightening press, and it would then be reasonable to expect them to show a higher value of elastic strength.

In the case of built columns, the effect of the process of machine-riveting is another outside influence on the condition of the material which requires recognition. Every practical constructor knows that in riveting up a member by hydraulic machine-riveters, the various parts have a tendency to stretch out and creep past each other, sometimes in very different degrees, resulting in the members twisting or bending out of a straight line, and no clearer evidence can be adduced to prove the existence of somewhat heavy internal stresses in the finished work.

In symmetrical sections the effect of the riveting down one side will be apparently neutralized by the subsequent riveting on the other; but in an assemblage of plates and angle bars, or other sections, as already remarked, the plates and bars often stretch or creep in different degrees, so that, although the resulting member may be quite straight and free from twist, this will be no proof of the non-existence of severe artificial internal stresses.

Among the instances of this which have occurred in the writer's experience, one, in connection with a bridge over the River Tyne, England, may be mentioned.

In order to guard against this creeping tendency, the writer specified that the large columns of the river piers should have their butt-joint ends machined over the full section, after having been riveted up in lengths in the contractors' yard, to ensure that the butts should bear on each other for the full sectional area of the members.

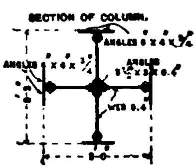


FIG. 1.

These columns are of cruciform section, as shown in Fig. 1, and the total length of each consists of three lengths of about 27 ft. By some oversight, however, in the case of some of the columns, the specified requirement was not carried out in the contractors' yard, and instead, the ends of each individual bar and plate were machined to a very good fit before rivet-

ing was commenced. Tacking rivets were then put in to hold the various parts in their correct relative positions, and the riveting was proceeded with throughout the length of 27 ft., and on its completion it was found that the angle bars and plates had crept past each other in varying degrees, so that the previous careful fitting was of no avail, and the joint had to be re-dressed by hand. The web-plate ends were also found to be hollow to the extent of $\frac{1}{16}$ in. in the half-width of column between the angle bars, and this undoubtedly points to the fact that internal stresses must exist in the column as made.

The columns appeared to be quite straight and true as a whole, and the material of the plates and angles was all made and tested under the same specified requirements of tensile strength, 28 to 32 tons (2 240 lbs.) per square inch, with ultimate elongation of at least 20% in 8 ins. The material is open-hearth acid-process steel, throughout.

In the case of cast-iron columns, of course, cold-straightening or machine-riveters do not enter as disturbing influences, but it is hardly necessary to point out the probably similar influences due to the hidden defects and internal stresses known to exist more or less in all castings.

The other influences to which the writer has referred, *i. e.*, the presence of initial curvature or small errors in setting specimens in the testing machine, have by no means an insignificant value in the results obtained in experiments on "centrally" loaded columns.

Mr. James Christie*, in his paper entitled "Experiments on the Strength of Wrought-Iron Struts," remarked that his specimens were practically straight, but careful measurements revealed the existence of small amounts of initial curvature. These measurements are recorded in his tables of results, and the calculations of deflection hereafter given will show that they had an appreciable influence on the strength of the specimens. Mr. Charles A. Marshall's tests also give valuable evidence in the same direction.

The importance of apparently very small errors in setting the columns in the testing machine also received most definite practical demonstration in the tests by Mr. Christie and Mr. Marshall, and also in those by Professor Bauschinger, since moving the specimens very small distances from their original positions had great influence on the results.

* *Transactions*, Am. Soc. C. E., Vol. xiii.

In view of these prejudicial influences, it can be no matter for surprise that such wide differences exist between results obtained under apparently identical conditions, by the same experimenter, on the same material and with columns of precisely the same proportions. It is surprising, however, that attempts are made constantly to express the average results of such wide differences by a single-line formula.

What would be thought of an engineer who used wrought iron and mild steel indiscriminately, in a bridge or other structure, after striking an average between the tensile strengths of the two materials as a basis upon which to determine the sections of the tension members? This would be no more erroneous and misleading than the present practice of using the average strength of columns of any particular class of material and of any one value of such proportions as are usual in practice. It is surely a much more rational and scientific procedure to ascertain within what limits we may reasonably expect the strength of columns to lie, and then to base our estimate of probable strength on the lower limit so derived.

No reference is here intended to be made to what would be deemed defective columns, in any respect, but only to columns which are in every practical sense believed to be above suspicion. The enunciation of the principle that the strength of columns of any given material cannot be represented by any single-line formula, but must be expressed by an area within which the results of experiments may be expected to lie, was, the writer believes, first made by Professor T. Claxton Fidler, M. Inst. C. E.,* ascribing the variations in column strength to variations in the modulus of elasticity.

In plotting the results of the tests of various experiments upon the diagrams accompanying this paper, the writer has endeavored to show every test wherever possible, provided no defects or special circumstances were involved.

In the case of Professor Tetmajer's tests, the records of a considerable number of the results, as given in his tables, were the averages of two specimens, so that those diagrams on which Tetmajer's tests are plotted do not show the full range of the tests, and the differences between maximum and minimum results would be actually somewhat more than shown. In all other cases, however, it is believed that every result plotted on the diagrams refers to a single experiment.

* "On the Practical Strength of Columns, and of Braced Struts," *Minutes of Proceedings*, Institution of Civil Engineers, Vol. lxxxvi (1886).

As far as the writer is aware, Hodgkinson's tests of columns of Low Moor, No. 3, cast iron and of wrought iron, with both ends rounded or both ends flat, are shown here complete and in their full number (within the limits of length of the diagrams) for the first time.

Hodgkinson's other experiments on cast-iron columns are also shown on a diagram to which further reference will be made, but these tests were on specimens of various kinds of cast iron, and by themselves could form no reliable guide, although, together with Hodgkinson's experiments on Low Moor, No. 3, cast iron, they have formed practically the only basis for the design of cast-iron columns for the last fifty years; and extensive tables of the strength of cast-iron columns, based on them, and calculated from Gordon's or Rankin's formulas, are to be found in nearly every pocket-book published for the use of engineers.

The diagrams have all been plotted to such proportions as to show clearly the differences between the various tests and also their relation to the calculated curves derived from the writer's formulas. It would have been easy to have shown an apparently better agreement by adopting other proportions for the diagrams.

The writer proposes to deal with the theory and resulting formulas in the first place, and afterward to compare them with the results of nearly all the more important series of tests hitherto made upon columns of cast iron, wrought iron, steel and timber.

Theoretical Principles.

Any column with the smallest eccentricity of loading or the smallest amount of initial curvature will immediately begin to deflect under load, and the deflection will increase in a much more rapid degree than the increase of load, and every increase in deflection tends still further to increase deflection. It is this last fact which makes a column of moderate length so exceedingly sensitive to small deviations in loading or to small initial bends.

In order to arrive at an expression for the strength of any column, it is therefore necessary to develop first a formula to express the deflection of that column under given conditions. The deflection of a column is caused solely by the bending moments imposed upon it, and the laws of deflection are therefore the same as for a beam under transverse stress.

The probable deflection of a solid beam subjected to bending moments can be determined very simply, with all necessary practical accuracy, if the relations existing between the stress diagram and the resulting deflection are made use of.*

These relations apply with equal correctness to the case of a column. Let $P\ Q$ in Fig. 2 represent a cantilever of uniform section, and let $P\ Q\ R$ be the diagram of bending moments—either regular or irregular in outline. Let $I =$ moment of inertia of the cross-section of the cantilever, and $E =$ modulus of elasticity of the material used.

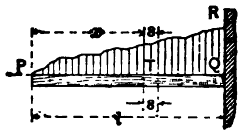


FIG. 2.

Take sections at distances x and $x + s$ from the point P at the extreme outer end of the cantilever, and let it be required to find the deflection of the point P below the point T at the center of s, caused by the stress existing between the two sections, which are further supposed to be exceedingly close together, so that s is very small compared to x. This being so, the bending moment on the cantilever between the two sections may, without appreciable error, be assumed as uniform for the length s.

Then, if M be the bending moment at the sections considered, and c, c' the distances of the extreme fibers from the neutral axis of the sections, the resulting stress in the fibers will be

$$f = \frac{Mc}{I} \text{ and } f' = \frac{Mc'}{I},$$

the common formula for the stress in a solid beam.

The resulting extension—or compression—of these extreme fibers will be $\lambda = \frac{fs}{E}$ and $\lambda' = \frac{f's}{E}$, and the point P will be deflected below the point T, as shown in Fig. 3 by dotted lines, and it is clear that if we reduce s to an exceedingly small quantity $\frac{\delta}{x} = \frac{\lambda}{c} = \frac{\lambda'}{c'}$, since the angle subtended by δ is very small; and therefore

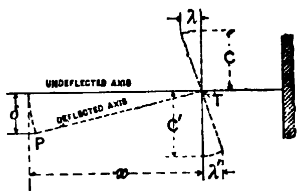


FIG. 3.

$$\delta = \frac{\lambda x}{c} = \frac{fsx}{Ec} = \frac{Mcsx}{IEc} = \frac{Msx}{IE}.$$

* "Theory of Solid and Braced Elastic Arches." by W. Cain, M. Am. Soc. C. E., 1879; and "Continuous-Girder Bridges," by T. C. Fidler, *Minutes of Proceedings*, Institution of Civil Engineers, Vol. lxxiv, 1883.

We have here dealt only with the deflections resulting from the bending moment and stress existing between the two sections, at distances x and $x + s$ from the point P, but the same relation holds for all other sections, and the total deflection caused by the bending moments along the whole length l of the cantilever will be $\Sigma\, \delta = \varDelta =$ the sum of the values of $\frac{M s x}{E I}$ between the points P and $Q = \Sigma \frac{M s x}{E I}$.

The numerator of this quantity is simply the moment of the area of the diagram of bending moments around the extreme point P, and the equation may be put into the form

$$\varDelta = \frac{A\, X}{E I},$$

where A = area of diagram of bending moments,

X = distance of center of gravity of this area from the point P.

It is to be remembered that the deflection to be dealt with in beams is always practically very small, compared to the length of the beam, and the above reasoning is not intended to be applied to absurd and improbable extremes. The relations deduced would not, of course, apply to a girder with a deep braced web under heavy shearing stresses, but on a solid beam section the deflection due to shearing stress is very small compared to that due to transverse bending; and, in the case of columns, whether solid or with braced webs, the shearing stresses are again much less than are usual in beams.

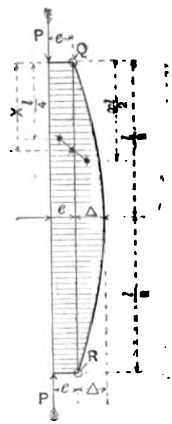


FIG. 4.

It is a simple matter to apply the foregoing relations to the case of a column under eccentric load, and with round or perfectly free pivoted ends. Let $Q\, R$ (Fig. 4) represent the axis of a column of length l acted upon by forces P acting at a distance e from the axis at its ends, with a resulting deflection $\varDelta$ at the center of the column's length.

The column deflection is exaggerated in the figure, for the sake of clearness, and the length l marked thereon appears to be considerably shorter than the true length of the curved axis of the bent column, but the difference in an actual column under test would be very small, and would not affect the reasoning to any appreciable extent.

The diagram of bending moments will then be as shown in the

figure included between the curve of the bent column and the line of action of the end forces $P\,P$, and, treating each half of the column as a cantilever, the deflection from the tangent to the bent column at its center will be $\varDelta = \frac{A\,X}{E\,I}$ when equilibrium is established between the bending moments produced by the eccentric load with the resulting deflection and the internal moment of resistance of the column sections.

In order to solve this equation for any given case, it is necessary, theoretically, to know the exact character of the curve of the bent column, as the bending moment at any point $= P\,(e + \varDelta')$, where $\varDelta'$ is the deflection from the chord line at that point, but we are mainly concerned with the deflection and bending moment at the point where these are maxima, *i. e.*, at the center of the column length, and the central deflection is the very quantity that is sought.

Practically, however, it is of small importance to know the precise nature of the curve, and a considerable divergence from theoretic accuracy in our knowledge of its true character makes but a trifling difference to the final result, as may be proved easily by assuming various outlines for the curve.

If the bending moment were perfectly uniform for the full length of the column, the curve taken by the latter would be part of a circle, and if the eccentricity of loading were indefinitely small, the curve would be a curve of sines. The actual curve is somewhere between these, and depends on the amount of eccentricity, and, as the deflection of columns in actual test or practice is very small in comparison with the column length, it is sufficiently accurate for all practical purposes to assume the curve to be a parabola, which will, under actual conditions, differ in an exceedingly small degree from the curves of both of the extreme conditions mentioned.

Under any circumstances, the area of the diagram of bending moments for the half length of a column, loaded as shown in Fig. 4, will be

$$A = \left(P \times e \times \frac{l}{2}\right) + \left(P \times \frac{\varDelta\,y\,l}{2}\right),$$

when y is a coefficient expressing the mean deflection in terms of the maximum deflection $\varDelta$ at the center of the column length, and the moment of this area around the extreme end Q of the column will be

$$A\,X = \overline{\left(P \times e \times \frac{l}{2}\right)\frac{l}{4}} + \overline{\left(P \times \frac{\varDelta\,y\,l}{2}\right) \times \frac{x\,l}{2}},$$

where x is a coefficient in terms of $\frac{l}{2}$, expressing the distance from Q of the center of gravity of the portion of the moment diagram included between the chord and the curve of the bent column, *i. e.*, that portion of the moment diagram due to column deflection.

Then
$$A\,X = \frac{P\,l^2\,e}{8} + \frac{P\,l^2\,\Delta\,y\,x}{4}$$
$$= \frac{P\,l^2}{8}(e + 2\,\Delta\,y\,x),$$

and
$$\frac{A\,X}{E\,I} = \Delta = \frac{P\,l^2}{8\,E\,I}(e + 2\,\Delta\,y\,x);$$

and therefore
$$8\,E\,I\,\Delta = P\,l^2\,(e + 2\,\Delta\,y\,x),$$

from which
$$\Delta = \frac{P\,l^2\,e}{8\,E\,I - 2\,P\,l^2\,y\,x} \quad\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Now, making use of the assumption that the curve of the bent column is a parabola, the corresponding values of y and x will be
$$y = \frac{2}{3}, \text{ and } x = \frac{5}{8};$$

and substituting these values in General Equation (1), we have as a practical result for the deflection of columns under eccentric load
$$\Delta = \frac{P\,l^2\,e}{8\,E\,I - (2\,P\,l^2 \times \frac{2}{3} \times \frac{5}{8})} = \frac{P\,l^2\,e}{8\,E\,I - \frac{5}{6}\,P\,l^2} \ldots\ldots \quad (2)$$

An inspection of these formulas at once shows that any column, of whatever material, with both ends round, and with the eccentricity of loading reduced to an exceedingly small degree, in fact, to as small an amount as we can form any conception of, so long as it has a positive value, would tend to have an infinite deflection, and therefore fail absolutely, as soon as the denominator of the right-hand member of the equation becomes zero.

Then, using Equation (1), Δ would have an infinite value when
$$8\,E\,I = 2\,P\,l^2\,y\,x;$$

and the ultimate load would therefore be
$$P = \frac{8\,E\,I}{2\,l^2\,y\,x}.$$

Under the assumed ideal condition of perfect central loading, the curve of the column when bent being a curve of sines, the values y

and x would each be $\frac{2}{\pi}$, and substituting these in the equation, we have as the ultimate load of the ideal column with both ends round

$$P = \frac{\pi^2 E I}{l^2} = \frac{9.87 E I}{l^2}$$

or Euler's formula.

If Equation (2), based on the curve of the column being assumed to be a parabola, were used in this way to estimate the ultimate load, instead of General Equation (1), with the correct values of y and x, the result would be

$$P = \frac{9.6 E I}{l^2},$$

so that by applying the assumption as to the curve of the column being a parabola, even to the ideal extreme, we would make an error on the safe side of only 2.73% below the theoretic truth in the estimate of ultimate load, and a small amount of eccentricity, such as we may reasonably expect in presumably centrally loaded columns in actual practice, would reduce this already small error to still smaller and practically inappreciable dimensions.

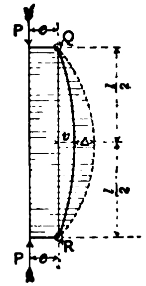


FIG. 5.

It is clear, therefore, from the foregoing, that the formulas for deflection, whether in the general form (1), or in the suggested practical form (2), are based on correct theoretic principles, and are of correct form.

We have dealt hitherto with the case of a column supposed to be absolutely straight before loading, and it remains to be seen what influence a small amount of initial curvature would have on the deflection.

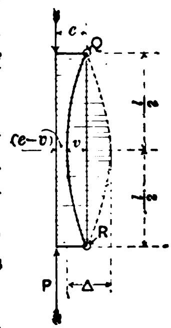


FIG. 6.

Let v, Figs. 5 and 6, represent the versine of an initial curvature, whether outwardly visible or not, in the axis of an eccentrically loaded column. It may have positive value, Fig. 5, or negative value, Fig. 6, relatively to the eccentricity e with which the load is imposed.

Let y_1 and x_1 be functions with regard to the area enclosed between the initial curve of the column and its chord line, of similar character to y and x, already adopted with regard to the curve of the column resulting from stress, or let y_1 and x_1 bear similar relations to v and l as y and x bear to Δ and l.

Then, whatever be the precise character of the initial curve and the deflection curve, using the same reasoning as before, we have

$$\frac{A\,X}{E\,I} = \frac{P\,l^2\,(e + 2\,y\,x\,\Delta \pm 2\,y_1\,x_1\,v)}{8\,E\,I} = \Delta$$

= central deflection due to stress,

and therefore $\Delta = \dfrac{P\,l^2\,(e \pm 2\,y_1\,x_1\,v)}{8\,E\,I - 2\,P\,l^2\,y\,x}$ .... (3)

This would give a minus value to Δ, if the quantity $2\,y_1\,x_1\,v$ should happen to have the minus sign, and at the same time be greater in value than e, but this would simply mean that the deflection would take place in the opposite direction to that in which e alone would cause it to bend. It must be kept in view that we are dealing with small amounts of initial curvature, shown in exaggeration in Figs. 5 and 6, for the sake of clearness.

In the case of Fig. 5, where the initial curvature is positive with regard to the eccentricity, *i. e.*, acts with the eccentricity to increase the deflection, the diagram of bending moments increases with the load imposed from the area bounded by the straight line joining the end forces $P\,P$, and the initial curve of the column to the area bounded by the line joining $P\,P$ and the curve of the deflected column.

The bending moment at the center of the column length is primarily $P\,(e + v)$, increasing, as the column deflects under load, to $P\,(e + v + \Delta)$. In the case of Fig. 6, the conditions are altered by the initial curvature acting against the eccentricity, and the primary bending moment at the center of the column length will be $P\,(e - v)$, increasing to $P\,(e - v + \Delta)$, or $P\,(e - v - \Delta)$, depending on whether the influence of e or v happens to be greater in producing the deflection Δ, since Δ must of necessity take the sign of the more influential of these quantities.

Now comparing Formula (3) with Formula (1), it will be seen that an initial curvature has an influence similar to an equivalent value of eccentricity of loading, and the term $(e \pm 2\,y_1\,x_1\,v)$ may be replaced by a single term represented by ε in those columns for which we cannot determine precisely the correct value e and v, such as the presumably centrally loaded column of ordinary material.

Substituting the symbol ε for e in Formula (2), we then have

$$\Delta = \frac{P\,l^2\,\varepsilon}{8\,E\,I - \frac{5}{6}\,P\,l^2} \text{........................(4)}$$

for the deflection of practical columns apparently straight, and apparently centrally loaded.

It must at once be recognized that it is practically impossible to assign any value for ε beforehand, for any particular column, for reasons already given.

The foregoing reasoning will explain how it may easily happen that two columns of identical dimensions and of identical material, as far as we are able to determine, might give very different results in the testing machine, by one having the internal v acting with the accidental e, and the other having its internal v acting contrary to its accidental e, since the value of e might be very appreciable in the first case, and in the second case, if $2\,y_1\,x_1\,v$ happened to be equal to $+\,e$, the value of ε would be zero, and the column would probably show a high ultimate strength.

We are now in a position to deduce a formula to express the maximum stress in a column under eccentric load, and to extend it to the case of the practical column under presumably central load.

Referring to Fig. 4, the bending moment at the center of the column length is

$$M = P\,(e + \Delta) = \frac{I f_b}{c} = \frac{a\,r^2 f_b}{c},$$

and therefore $$f_b = \pm\,\frac{P\,(e + \Delta)\,c}{a\,r^2},$$

where r is the radius of gyration of the column section in the direction in which the column bends, and f_b represents the unit stress caused by the bending moment alone at a distance c from the neutral axis, and a = the sectional area of the column.

The direct compression on the column section at the same time is $+f_a = \dfrac{P}{a}$ = average load per square inch on the sectional area of the column, and the total stress in the extreme fibers will therefore be

$$F = \pm f_b + f_a = \pm\,\frac{P\,(e + \Delta)\,c}{a\,r^2} + \frac{P}{a},$$

and now, substituting the value of Δ from Equation (2),

$$F = \frac{P}{a} \pm \left(\frac{P\left(e + \dfrac{P\,l^2\,e}{8\,E I - \frac{5}{6}\,P\,l^2}\right)c}{a\,r^2}\right) =$$

$$\frac{P}{a}\left\{1 \pm \frac{c}{r^2}\left(\frac{P\,l^2\,e}{8\,E I - \frac{5}{6}\,P\,l^2} + e\right)\right\}$$

$$= f_a\left\{1 \pm \frac{c\,e}{r^2}\left(\frac{f_a\,l^2}{8\,E\,r^2 - \frac{5}{6}\,f_a\,l^2} + 1\right)\right\} \ldots\ldots\ldots\ldots\ldots (5)$$

From Equation (5) it is probable that an expression may be deduced to give the average load per square inch f_a corresponding to a given maximum of minimum stress F with varying values of the other factors, but the writer has found it much simpler and easier to deal with the value of the ratio $\frac{l}{r}$ corresponding to a given value of maximum or minimum stress F, average stress, f_d, modulus of elasticity E, and a given value of $\frac{c\,e}{r^2}$.

Efforts have been made, in connection with most column formulas, to determine the value of f_a for a given value of $\frac{l}{r}$, but the writer has never found, in his own practice, any advantage in this, and it is equally convenient to be able to determine the value of $\frac{l}{r}$ corresponding to a given value of f_a. Either way is, as a matter of practice, equally suitable for the purpose of laying down a curve to express the strength of varying proportions of columns.

From Equation (5), the general expression for the maximum stress produced in a column, we have

$$F = f_a \left\{ 1 \pm \frac{c\,e}{r^2} \left(\frac{f_a\, l^2}{8\, E\, r^2 - \frac{5}{6} f_a\, l^2} + 1 \right) \right\}$$

and, dividing the factor $\dfrac{f_a\, l^2}{8\, E\, r^2 - \frac{5}{6} f_a\, l^2}$ by r^2,

$$F = f_a \left\{ 1 \pm \frac{c\,e}{r^2} \left(\frac{f_a \frac{l^2}{r^2}}{8\, E - \frac{5}{6} f_a \frac{l^2}{r^2}} + 1 \right) \right\},$$

and using the symbol R to represent $\dfrac{l}{r}$, we have

$$F = f_a \left\{ 1 \pm \frac{c\,e}{r^2} \left(\frac{f_a\, R^2}{8\, E - \frac{5}{6} f_a\, R^2} + 1 \right) \right\}$$

$$= f_a \left\{ 1 \pm \frac{c\,e}{r^2} \left(\frac{f_a\, R^2 + 8\, E - \frac{5}{6} f_a\, R^2}{8\, E - \frac{5}{6} f_a\, R^2} \right) \right\}$$

$$= f_a \left\{ 1 \pm \frac{c\,e}{r^2} \left(\frac{48\, E + f_a\, R^2}{48\, E - 5 f_a R^2} \right) \right\} \qquad \text{(6)}$$

and now using the + sign in the brackets to determine the maximum fiber stress F_c

$$F_c = f_a \left(1 + \frac{c\,e}{r^2} \left\{ \frac{48\, E + f_a\, R^2}{48\, E - 5\, f_a\, R^2} \right\} \right)$$

from which we have, by simple algebraic transformation, the value of R corresponding to any fixed value of the maximum compressive stress F_c,

$$\text{or } R = \sqrt{\frac{48\,E}{5\,F_c + f_d\left(\frac{c\,e}{r^2} - 5\right)}\left[\frac{F_c}{f_d} - 1 - \frac{c\,e}{r^2}\right]} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

Similarly, using the — sign in the brackets in Equation (6) to determine the minimum fiber stress (not necessarily tensile) F_t.

$$F_t = f_d\left\{1 - \frac{c\,e}{r^2}\left(\frac{48\,E + f_d\,R^2}{48\,E - 5\,f_d\,R^2}\right)\right\}$$

which is easily transformed to

$$R = \sqrt{\frac{48\,E}{5\,F_t - f_d\left(\frac{c\,e}{r^2} + 5\right)}\left[\frac{F_t}{f_d} - 1 + \frac{c\,e}{r^2}\right]} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

It should be noted here that F_t only becomes tensile when the maximum tensile fiber stress f_b caused by bending is greater than the direct compressive stress f_d, and it should also be noted that F_t must be given its proper sign to correspond with its character, *i. e.*, + when compressive and — when tensile, irrespective of the fixed signs shown in Equation (8).

The precise use of these equations (7) and (8) is as follows:

Let it be assumed that in a given section of column, we decide that a certain value of maximum compressive stress F_c, or a certain value of minimum stress F_t, is not to be exceeded; these values being inserted in Formulas (7) and (8), together with the value of E and $\frac{c\,e}{r^2}$, corresponding to the material used and the section of column and eccentricity of loading actually adopted, we have at once the value of R corresponding to different values of f_d, the direct load per square inch.

Both of these formulas reduce to exceedingly simple terms on the insertion of the physical constants E, F_c, or F_t, and the proper value of $\frac{c\,e}{r^2}$, as will be seen later in their application.

Formulas (4), (5), (6), (7) and (8) are all general expressions applicable to the case of columns with both ends free, and of any given material and form of section, and with any given value of eccentricity of loading probable in practical work.

Professor William Cain* came to the conclusion that, with an ideal column, perfectly centrally loaded, up to the value given by Euler's

* "Theory of the Ideal Column," *Transactions*, Am Soc. C. E., Vol. xxxix.

formula, a very small increase to this load insures failure of the column. A very similar conclusion can be drawn by applying Formula (4) to a given example, say $E = 30\,000\,000$, $I = 10$ ins.^{4}, $l = 300$ ins., and assume $e = 0.001$ in., corresponding to an accuracy of loading far beyond practical possibilities.

$$\text{Then } \Delta = \frac{Pl^2 e}{8\,EI - \frac{5}{8}Pl^2} = \frac{P \times 90\,000 \times \frac{1}{1000}}{2\,400\,000\,000 - 75\,000\,P}$$

and if P be taken in units of 10 000 lbs., this reduces to

$$\Delta = \frac{12\,P}{32\,000 - 10\,000\,P}.$$

Working this out for the various values of P, we have the results shown in Table No. 1.

TABLE No. 1.

P, in pounds.	Δ, in inches.	P, in pounds.	Δ, in inches.	P, in pounds.	Δ, in inches.
0	0.0000	31 600	0.0948	31 970	1.2788
20 000	0.002	31 650	0.1085	31 975	1.5348
25 000	0.0043	31 700	0.1268	31 980	1.9188
28 000	0.0084	31 750	0.1524	31 985	2.5588
29 000	0.0116	31 800	0.1908	31 990	3.8388
30 000	0.018	31 850	0.2548	31 992	4.7988
30 500	0.0244	31 900	0.3828	31 994	6.3988
31 000	0.0372	31 950	0.7668	31 996	9.599
31 500	0.0756	31 960	0.9588	32 000	∞

These results are instructive, and it hardly needs a calculation of maximum fiber stress to show how great is the effect of the small additions near to the ultimate load of 32 000 lbs.

The value of I assumed $= 10$ ins.$^4 = a\,r^2$, and nearly corresponds to a rectangular solid section 3.307 ins. square, with area nearly 10.94 sq. ins., and least radius of gyration $= 0.955$ in. nearly, so that the ratio $\frac{l}{r} = \frac{300}{0.955} = 314$.

The maximum compressive fiber stress

$$F_c = \frac{P\,(e + \Delta)\,c}{a\,r^2} + \frac{P}{a} = \frac{P\,(0.001 + \Delta) \times 1.6535}{10} + \frac{P}{10.94 \text{ sq. ins.}}$$

from which we have, when

$P =$	$\Delta + e =$	F lbs. per square inch $=$
31 990	3.8398	23 200
31 992	4.7998	28 300
31 994	6.3998	37 000
31 996	9.6	53 600
32 000	∞	∞

A most interesting feature of these figures for maximum fiber stress is the theoretic assurance which they give as to the capacity of long columns to resist fatigue, even when loaded nearly up to the crippling point; and if the material of the column dealt with in the example be assumed to have a compressive elastic limit of, say, 40 000 lbs. per square inch, it will be seen that the column would be quite uninjured by an infinite number of loadings within 10 lbs. of its ultimate supporting power. This, of course, would only hold good if the load were imposed without the slightest dynamic effect, or impact.

This completes the investigation of what may be termed "the elementary column," of which, columns with fixed ends, flat ends and pin ends, may be considered as merely modifications.

It will be noticed that the formulas in each case include a term $\frac{c}{r}$ dependent on the form of column section, and the writer at one time hoped to find practical verification of the influence of form of section in the published results of experiments, but the influence of other factors is too great and the number of tests on any one form of section is too small to enable this to be done as yet.

Again, the number of experiments carried out with a value of eccentricity sufficiently great to make it a paramount factor is very small, and in the great mass of tests hitherto made, the endeavor has been to impose the load "centrally"; we must, therefore, substitute ε (see page 333) for e in the formulas when applying them to experiments under presumably central loads. Under these circumstances, it appears justifiable, in our present state of knowledge, to consider the factor $\frac{c\,\varepsilon}{r^2}$ as a constant, of which the value for centrally loaded columns must be determined from available experimental records.

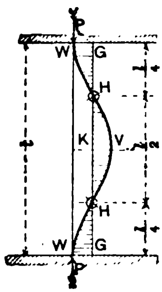


FIG. 7.

Attention will be given, next, to the fixed-ended column shown in Fig. 7, the section being assumed to be uniform, as before. When under load the column $W\,W$ will bend in a double reverse curve similar to that shown in the figure, and the central portion of the column $H\,H$ will behave similarly to, and be subject to, the same laws as a free-ended column. The question to be solved, in the first place, is as to the proportion of the total length of a fixed-ended column, which will act

as a free-ended column. This proportion is frequently stated to be one-half, without any reasons being stated.

In determining this it is convenient to consider one-half of the column only (since the other will necessarily behave in a precisely similar manner) as shown in Fig. 8, to a larger scale.

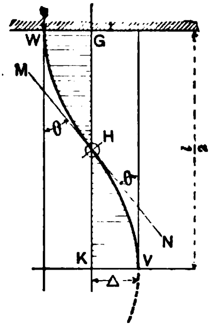


FIG. 8.

Referring again to Fig. 3 and its descriptive context it was shown that

$$\delta = \frac{M\,s\,x}{E\,I}$$

and therefore $\frac{\delta}{x} = \frac{M\,s}{E\,I}$ = the tangent of the angle of slope set up at the end of the length x by the stresses in the portion s of the cantilever considered; and as this angle of slope is in practice exceedingly small, its tangent will practically represent the angle in circular measure with all necessary accuracy, and the sum of all the exceedingly small angles of slope, for the full length of the cantilever, will then be

$$\Sigma\,\frac{\delta}{x} = \Sigma\,\frac{M\,s}{EI} = \frac{A}{E\,I} = \frac{\Delta}{X} = \Theta,$$

or the angle of slope at the extreme end of the cantilever is proportional to the area of the curve of bending moments.

As before, this reasoning and its results apply equally well to the bent column.

In Fig. 8 the tangent $M\,N$ to the curve of the bent column at the point of contrary flexure H is common to both portions of the curve, and the slope of each portion is, therefore, the same at this point, since the tangents at W and V remain vertical and parallel to each other, in consequence of the fixity of the end W and the symmetry of the whole column around the point V. The area of the bending moment diagram $G\,H\,W$ must, therefore, be equal to the area of the bending moment diagram $K\,H\,V$. Further, as the column section is assumed to be uniform, and there is no bending moment at the point H, where the two portions $W\,H$ and $H\,V$ react upon each other in simple compression and shear, the curvature of the two portions at corresponding points on either side of H must evidently be identical, inasmuch as both are of the same section, subject to the same forces and subject to the same laws of flexure.

From these conditions of equal curvature and equal area of bending moment diagram, it results that the length $G\,H$ must be equal to the length $K\,H$, and each of them will, therefore, be one-fourth of the total length of the column. Also the portion $W\,G$ of the total deflection must equal the portion $K\,V$, and the bending moment at the fixed ends W will equal the bending moment at the center V.

It is thus determined that the length of $H\,H$, Fig. 7, acting as a free-ended column, is one-half the total length L of the fixed-ended column, or, in other words, a fixed-ended column carrying a given load is twice the length of a free-ended column of the same section and having similar stresses.

This result is based on assumptions of perfect straightness before bending, perfectly homogeneous material, and perfect fixity of ends. In practical work some divergence will undoubtedly occur, which will require to be allowed for by an assumed equivalent eccentricity of loading, as in the case of the simple free-ended column already dealt with, and we therefore have, for the fixed-ended column:

$$R = 2\sqrt{\frac{48\,E}{5\,F_c + f_d\left(\frac{c\,\varepsilon}{r^2} - 5\right)}\left[\frac{F_c}{f_d} - 1 - \frac{c\,\varepsilon}{r^2}\right]} \quad \text{for failure by compression,}$$

$$\text{or } 2\sqrt{\frac{48\,E}{5\,F_t - f_d\left(\frac{c\,\varepsilon}{r^2} + 5\right)}\left[\frac{F_t}{f_d} - 1 + \frac{c\,\varepsilon}{r^2}\right]} \quad \text{for failure by tension.}$$

In actual practice the true fixed-ended column rarely, if ever, exists. It is difficult, even in experiments in a testing machine, to comply with the conditions necessary to ensure absolute fixity of ends, and in ordinary construction the difficulty is increased greatly.

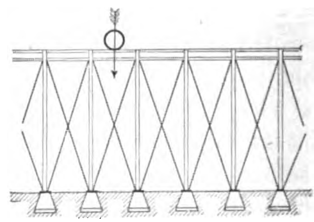


FIG. 9.

The writer has found that the vaguest ideas are sometimes held as to what is required to realize fixed ends in a column. A consideration of simple examples will probably exhibit this matter in a clearer light.

Assume, in the first instance, that we have, as in Fig. 9, a series of stiff gantry girders, 2 ft. deep by 10 ft. span, riveted securely to the heads of columns 30 ft. high, firmly braced together to preserve their

verticality. Assume, also, that the foundation blocks on which the column rests are very rigid, that the columns have large well-bolted bases, and that the ratio $\frac{l}{r}$ of these columns is very large, and the columns therefore slender in proportion.

Then the imposition of load on any span will cause deflection in the girder, and the ends of the girder will deviate from the vertical to a slight degree, but the relative stiffness of the girders themselves, as compared with the column, being high, the approximation to ideal fixity of ends would, practically speaking, be of a high degree.

In the second instance, Fig. 10, let the columns be spaced at 30 ft. centers, retaining the same depth of girder, 2 ft., and merely increasing the girder sections to obtain the same value of working unit stress, while increasing the radius of gyration of the columns to provide much greater stiffness of column. Under these conditions, the deflection of the girder under load, and consequently the slope of the ends of the girders where they are securely riveted to the column heads, would be increased largely, and the columns would be subjected to heavy bending stresses in addition to their direct load. These columns would be much less heavily stressed if they had pin-joint connections to the girders, and the apparent fixity of end, given by a secure riveted connection, would actually be accompanied by severely prejudicial secondary stresses.

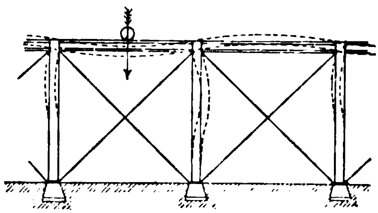


FIG. 10.

The conclusion derived from these examples is that in practical work the degree of approximation to fixity of ends depends entirely on the relative stiffness of the column and the other members of the structure attached to it; and the estimation of this degree of fixity demands the most careful consideration on the part of the engineer.

One of the advantages claimed for riveted connections in bridge work is that the compression members are thereby made into fixed-ended columns, and can be accorded higher stresses in consequence.

A portion of a riveted main girder of N-type is shown in Fig. 11, with connections of web members for two panel points, and the writer would ask whether the top boom fixes the ends of the vertical posts, or do the posts fix the ends of the panel lengths in the top boom (which is a column between panel points), or are we to rely upon the stiffness of the diagonal tension members to fix both?

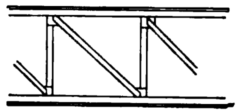


FIG. 11.

The last can hardly be considered a reasonable assumption, as the most heavily loaded portions of the top boom are at the center of the span, where also the lightest diagonals are found, and as regards the posts fixing the ends of the panel lengths of the top boom, this also is out of the question, as the stiffness of the posts is usually small as compared with the stiffness of the boom, and if we consider the top boom as fixing the ends of the vertical posts, under which class of columns are we to place the top boom panel lengths? They could not be considered as fixed-ended, and as they would have to perform the additional duty of fixing the vertical post ends, they could not be considered to be as favorably circumstanced as a round-ended or pivot-ended column. Here, again, we have to give consideration to relative stiffness of parts.

It is common knowledge that heavy secondary stresses exist in the connections of various members in a riveted structure, but it is not commonly recognized that these very secondary stresses may totally destroy any imaginary fixity of ends in the compression members, and actually place the members under worse conditions of stress than if pivoted end-bearings were adopted.

The writer is neither seeking to depreciate the practical value of the riveted connections nor to advocate either pin or pivoted end connections, but only wishes to point out the erroneous principles on which designs are frequently based.

As far as the writer is aware, no attempt has hitherto been made to arrive at a rational basis for the strength of flat-ended columns, although the greater number of tests of columns have been made with this class of end-bearing. As a rule, the assumption has been made that they act in precisely the same manner as fixed-ended columns, and column formulas to cover both in one expression are frequently given. This is quite erroneous, both from a theoretical point of view,

and from the evidence of actual experiments. With flat ends, no tensile stress can be developed at the ends, and with fixed ends it has been shown that the bending moment at each end is theoretically equal to that at the center of the column.

It is clear, then, that so long as no tensile stress is set up in a column with flat ends, it will behave as a fixed-ended column, that is, up to the point of loading at which the stress in the column at its ends and center is as shown in Fig. 12, when a small increase of load on the column will most probably cause failure.

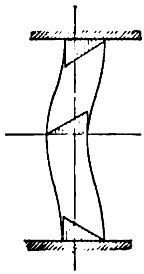


FIG. 12.

If, then, we use the formula for minimum stress F_t, and make F_t equal to zero, we obtain the value of R at which a given value of f_d will produce a minimum stress zero, and we will thus determine the value of R corresponding to incipient tensile stress, and the formula with F_t inserted as of zero value, will give what may be called the critical value,

$$R = 2\sqrt{\frac{48\,E}{0 - f_d\left(\frac{c\,\varepsilon}{r^2} + 5\right)}\left(0 - 1 + \frac{c\,\varepsilon}{r^2}\right)}$$

$$= 2\sqrt{\frac{48\,E\left(-1 + \frac{c\,\varepsilon}{r^2}\right)}{-f_d\left(\frac{c\,\varepsilon}{r^2} + 5\right)}}.$$

$$= 2\sqrt{\frac{48\,E\left(1 - \frac{c\,\varepsilon}{r^2}\right)}{f_d\left(\frac{c\,\varepsilon}{r^2} + 5\right)}} \quad \left\{\begin{array}{l}\text{when tension is incipi-}\\ \text{ent in flat-ended}\\ \text{columns.}\end{array}\right\} \ldots (9)$$

Here, again, it is necessary to determine from experimental results what value must be given to the factor $\frac{c\,\varepsilon}{r^2}$, if the formula is to be used to determine ultimate strength.

It may be urged that the load producing incipient tension in any given flat-ended column may not be the ultimate load; but, as soon as tension is attempted to be set up at the ends of a flat-ended column, the column will be in a highly unstable condition, and the ends will begin to rotate on their bearing faces. This is most readily seen by considering the case of a fixed-ended column in which tensile stresses have been set up at the ends and center.

If the fibers in tension at the ends were cut so as to transform the column into a flat-ended column while under load, we would naturally expect the column to alter its curvature immediately, and largely increase its deflection, with the result that it would probably fail immediately, or with a comparatively small additional load. The substantial truth of this, in practice, is most clearly evident in Mr. Christie's experiments, as will appear later.

By plotting the curves for the two conditions, one for failure by maximum compressive stress, and the other for the critical condition of incipient tension, it is made evident that with any given section of column, up to a certain value of R, dependent on the eccentricity of loading and modulus of elasticity, no tension can be set up in the column, whatever the load, and flat-ended columns below this limiting proportion behave in every sense as fixed-ended columns, while beyond this point the strength will fail more or less rapidly.

The writer believes that the value of the difference in the strength of fixed and flat-ended columns is here dealt with in a rational manner for the first time.

With regard to pin-ended columns, it is quite useless to theorize with the view of showing their superiority to round or pivot ends, owing to the fact that their behavior under load, even in a testing machine, depends very largely on the closeness of the fit between pin and hole, upon the smoothness or otherwise of the bearing surfaces, upon the diameter of the pin in relation to the radius of gyration, and upon the presence, either accidental or premeditated, of a lubricating medium.

In actual practice, the vibration in a railway bridge, caused by the passage of the load, and the movements of the members relatively to each other under the common variations of stress, must undoubtedly go very far to destroy the friction upon which depends the superiority with which this type of strut is often credited over those with round ends.

There is as yet no satisfactory and conclusive evidence that in practical work the pin-ended column can fairly be credited with this greater strength, and the practice of imposing higher stresses on account of the pin ends is open to grave question.

This matter may be viewed from another standpoint, that of the advantages claimed for the pin joint as compared with the riveted con-

nection. Among these so-called advantages are freedom from secondary stresses and greater certainty of realizing the ideal condition of centrality of loading on the various members.

Any additional strength accompanying the pin-bearing type of column can only be obtained when frictional resistances are set up in the bearing, preventing rotation, and thus bringing into play a moment of resistance to bending on the column end, and this moment of resistance in turn can only be developed by subjecting the other members assembled on the same pin to secondary bending stresses in order to realize a partial fixity of column ends.

In any case, the additional resistance due to partial fixity of ends in the pin-ended column, if it actually exists in practical construction, must be obtained at the expense of the other members on the same pin, and is largely dependent on the stiffness of those members. The question may fairly be raised whether or not it would be consistent practice to make an allowance for the secondary stresses in these other members, if we rely on these secondary stresses to provide the column with increased resistance.

Comparison of the Formulas with the Experiments.

Deflection Formula (4) for Round-Ended or Pivot-Ended Columns.

$$\Delta = \frac{P\, l^2\, \varepsilon}{8\, E\, I - \frac{5}{6}\, P\, l^2}.$$

The starting point for the whole of the foregoing theory and formulas was the development of this expression for the deflection of a column, and it has already been pointed out that, in practical work, some of the controlling influences cannot be made subject to actual observation, depending as they do on internal conditions arising from past history, accidental errors in setting, etc., etc.

Nevertheless, it is important to have some definite knowledge as to whether the deflection formula bears characteristic features having practical agreement with the results of actual observations in experiments.

In order to make a comparison it was necessary to make numerous trial calculations with different estimates of the values of those factors which are primarily unknown, and the process of comparison therefore consisted of fitting the calculated values given by the formula to the observed deflections, so that it might be seen whether curves

plotted with loads as abscissas and deflections as ordinates have the same character by calculation and observation.

The unknown factors for which the values have had to be estimated are the following:

1. The modulus of elasticity.................................... E,
2. The equivalent eccentricity................................ ε,
3. Any small amount of initial curvature capable of observation, V,

but not always noted in records of test and making the total deviation of the column from a straight line $= D = \Delta \pm V$.

It was found that the influence of each of these three factors is so great that very small deviations from the estimated values finally adopted destroy the agreement between calculation and observation, and as, in these estimated values, we are already dealing with very small quantities, it is to be noted that the small deviations referred to would be incapable of being observed in any ordinary experiment.

The examples selected for purposes of comparison have been taken from the tests of round-ended columns of Low Moor, No. 3 cast iron, made by Mr. Eaton Hodgkinson,* and from these tests of round-ended wrought-iron columns, made by Mr. James Christie.†

These examples have been chosen solely on account of the fullness of the records of deflections. In each case the effective column length has been taken as being the distance between the centers of the hemispherical ends, as shown in Fig. 13.

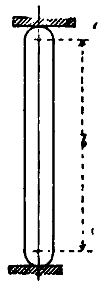


FIG. 13.

Examples from Hodgkinson's Tests of Low Moor, No. 3 Cast Iron.

Test No. 5, of Table I.‡—Solid, cylindrical, hemispherical-ended column, 60.50 ins. length over ends, 0.99 in. diameter, say 59.50 ins. effective length.

$$I = \text{moment of inertia of section} = 0.472 \text{ in.}^4$$

Estimated values: $\begin{cases} E = 14\,500\,000 \text{ lbs., modulus of elasticity;} \\ \varepsilon = 0.055 \text{ in., equivalent eccentricity of loading;} \\ V = +\,0.07 \text{ in., probable initial curvature escaping observation.} \end{cases}$

* Recorded in the *Philosophical Transactions* of the Royal Society of London, for 1840.

† *Transactions*, Am. Soc. C. E., Vol. xiii.

‡ *Philosophical Transactions*, Royal Society, London, 1840.

P = load, in pounds.	(Δ Inches.	+ V) Inches.	= D calculated. Inches.	D observed. Inches.
515....	0.0253	+ 0.07	= 0.0953..........	0.05
655....	0.0360	+ 0.07	= 0.1060..........	0.10
991....	0.0756	+ 0.07	= 0.1456..........	0.14
1 183....	0.1160	+ 0.07	= 0.1860..........	0.19
1 471....	0.2525	+ 0.07	= 0.3225..........	0.32
1 615....	0.4425	+ 0.07	= 0.5125..........	0.52
Ult. load. 1 663....	0.5690	+ 0.07	= 0.6390..........	α failed.

Test No. 20, of Table I.—Solid, cylindrical, hemispherical-ended column, 60.50 ins. length over all, 1.97 ins. diameter, say 58.50 ins. effective length.

$$I = 0.74 \text{ in.}^4$$

Estimated values: $E = 13\ 500\ 000$ lbs., $\varepsilon = 0.0675$ in., $V = 0$.

P = load, in pounds.	(Δ Inches.	+ V) Inches.	= D calculated. Inches.	D observed. Inches.
3 355....	0.0109	+ 0	= 0.0109...........	bent.
7 386....	0.0287	+ 0	= 0.0287..........	0.02
12 970....	0.0686	+ 0	= 0.0686...........	0.07
19 943....	0.1943	+ 0	= 0.1943...........	0.20
21 035....	0.2360	+ 0	= 0.2360...........	0.23
22 127....	0.2800	+ 0	= 0.2800...........	0.28
23 219....	0.3740	+ 0	= 0.3740...........	0.37
24 311....	0.5000	+ 0	= 0.5000...........	0.50 to 0.52
24 857....	0.5940	+ 0	= 0.5940...........	0.60
Ult. load. 25 403....	0.7230	+ 0	= 0.7230...........	8 failed.
(26 000)....	0.9350	+ 0	= 0.9350...........	....

Test No. 1, of Table VIII.—Hollow, cylindrical, hemispherical-ended column (Fig. 14), 90.75 ins. length over all, 1.78 ins. external diameter, 1.21 ins. internal diameter, say 89 ins. effective length.

Core center 0.19 in. out of center of external circle of column (ascertained after fracture). Center of area 0.09 in. out of center of external circle of column.

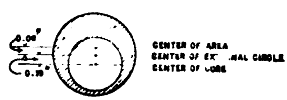


FIG. 14.

$$I = 0.367 \text{ in.}^4$$

Estimated values: $E = 15\ 000\ 000$ lbs., $\varepsilon = 0.11$ in., $V = -0.03$ in.

	P = load, in pounds.	(Δ Inches.	— V) Inches.	= D calculated. Inches.	D observed. Inches.
	2 237....	0.06655	— 0.03	= 0.03655........	0.03
	2 813....	0.09933	— 0.03	= 0.06933........	0.07
	3 317....	0.13050	— 0.03	= 0.10050........	0.11
	3 821....	0.17700	— 0.03	= 0.14700........	0.16
	4 325....	0.24330	— 0.03	= 0.21330	0.20
	4 829....	0.34600	— 0.03	= 0.31600........	0.32
	5 333....	0.52500	— 0.03	= 0.49500........	0.49
Maximum.	5 585....	0.67760	— 0.03	= 0.64760....	not observed; column not al-
	(6 000)....	1.17700	— 0.03	= 1.14700....	lowed to break.

Test No. 5, of Table VIII.—Hollow, cylindrical, hemispherical-ended column (Fig. 15), 90.75 ins. length over all, 2.23 ins. external diameter, 1.53 ins. internal diameter, therefore, say 88.52 ins. effective length.

Core center was 0.135 in. out of center of external circle of column (ascertained after fracture). Therefore center of area was 0.12 in. out of center of external circle.

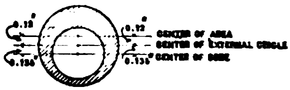


FIG. 15.

$$I = 0.8863 \text{ in.}^4$$

Estimated values: $E = 14\,500\,000$ lbs., $\varepsilon = 0.175$ in., $V = -0.025$ in.

	P = load, in pounds.	(Δ Inches.	— V) Inches.	= D calculated. Inches.	D observed. Inches.
	2 237....	0.0347	— 0.025	= 0.0097.......	0.04
	4 325....	0.0796	— 0.025	= 0.0546.......	0.06
	6 341....	0.1415	— 0.025	= 0.1165.......	0.12
	8 357....	0.2375	— 0.025	= 0.2125.......	0.22
	9 365....	0.3085	— 0.025	= 0.2835.......	0.28
	10 373....	0.4040	— 0.025	= 0.3790.......	0.37
	11 381....	0.5475	— 0.025	= 0.5225.......	0.55
	12 137....	0.7080	— 0.025	= 0.6830.......	0.69
Maximum.	12 389....	0.7525	— 0.025	= 0.7275....	not observed; column not al-
	(13 000)....	(0.9950)	— 0.025	= (0.9700)...	lowed to break.

Test No. 15, of Table VIII.—Hollow, cylindrical, hemispherical-ended column (Fig. 16), 90.75 ins. length over all, 3.36 ins. external diameter, 2.61 ins. internal diameter, say 87.39 ins. effective length = l.

Core center, after fracture, found to be 0.067 in. out of center of external circle of column. Therefore, center of area was 0.105 in. out of center of external circle.

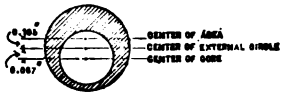


FIG. 16.

$$I = 3.915 \text{ ins.}^4$$

Estimated values: $E = 12\,000\,000$ lbs., $\varepsilon = 0.16$ in., $V = +0.035$ in.

P = load, in pounds.	(Δ Inches.	+ V) Inches.	= D calculated. Inches.	D observed. Inches.
3 355	0.0166	+ 0.035	= 0.0466	bent.
16 115	0.0727	+ 0.035	= 0.1077	0.09
18 667	0.0895	+ 0.035	= 0.1245	0.13
21 729	0.1128	+ 0.035	= 0.1478	0.15
24 148	0.1341	+ 0.035	= 0.1691	0.17
28 986	0.1872	+ 0.035	= 0.2222	0.24
33 824	0.2625	+ 0.035	= 0.2975	0.30
37 701	0.3445	+ 0.035	= 0.3795	0.38
41 632	0.4680	+ 0.035	= 0.5030	0.48
43 597	0.5550	+ 0.035	= 0.5900	0.59
45 563	0.6650	+ 0.035	= 0.7000	0.67
47 528	0.8175	+ 0.035	= 0.8525	0.87
48 511	0.9140	+ 0.035	= 0.9490	0.90
49 494	1.0320	+ 0.035	= 1.0670	1.07
Ult. load. 50 477	1.1750	+ 0.035	= 1.2100	∝

Test No. 7, of Table III.—Solid, rectangular pillar with hemispherical ends, 60.5 ins. length over all, 1.54 x 1.56 ins. nearly square, say 59 ins., effective length = l.

$$I = 0.475 \text{ in.}^4$$

Estimated values: $E = 13\,250\,000$ lbs., $\varepsilon = 0.06$ in., $V = -0.015$ in.

P = load, in pounds.	(Δ Inches.	— V) Inches.	= D calculated. Inches.	D observed. Inches.
2 141	0.010	— 0.015	= — 0.005	column bent.
4 465	0.025	— 0.015	= + 0.010	0.015
6 481	0.043	— 0.015	= + 0.028	0.02
11 169	0.130	— 0.015	= + 0.115	0.11
13 565	0.257	— 0.015	= 0.242	0.23
14 461	0.360	— 0.015	= 0.345	0.35
14 909	0.438	— 0.015	= 0.423	0.44
15 357	0.552	— 0.015	= 0.537	0.52
Ult. load. 15 581	0.632	— 0.015	= 0.617	∝

Examples from Christie's Tests of Wrought-Iron Struts with Hemispherical Ends.*

Test No. 204.—T-bar; 1 in. x 1 in. x 87.25 ins. length over all, 1-in. balls and plates, 86.25 ins. effective length $= l$.

$$I \text{ (least)} = a\,r^2 = 0.3 \text{ sq. in.} \times (0.26)^2.$$

Estimated values: $E = 23\,500\,000$ lbs., $\varepsilon = 0.0525$ in., $V = +0.025$ in.

	P = load, in pounds.	(Δ Inches.	+	V) Inches.	= D calculated. Inches.	D observed. Inches.
	100	0.0122	+	0.025	= 0.0372	0.05
	200	0.0304	+	0.025	= 0.0554	0.05
	300	0.0602	+	0.025	= 0.0852	0.08
	400	0.1178	+	0.025	= 0.1428	0.15
	500	0.2760	+	0.025	= 0.3010	0.30
Ult. load.	550	0.5410	+	0.025	= 0.5660	∞

Test No. 205.—T-bar; 3 ins. × 3 ins. × 82.0625 ins. length over all, 2-in. balls and plates, 80 0625 ins., effective length.

$$I = a\,r^2 = 2.53 \text{ sq. ins. x } (0.62)^2 = 0.9725 \text{ in.}^4$$

Estimated values: $E = 25\,000\,000$ lbs., $\varepsilon = 0.02$ in., $V = 0.035$ in.

	P = load, in pounds.	(Δ Inches.	+	V) Inches.	= D calculated. Inches.	D observed. Inches.
	500	0.0003	+	0.035	= 0.0353	0.03
	5 000	0.0038	+	0.035	= 0.0388	0.04
	10 000	0.0091	+	0.035	= 0.0441	0.05
	15 000	0.0168	+	0.035	= 0.0518	0.05
	20 000	0.0293	+	0.035	= 0.0643	0.06
	25 000	0.0526	+	0.035	= 0.0876	0.09
	30 000	0.1122	+	0.035	= 0.1472	0.15
	34 000	0.3380	+	0.035	= 0.3730	not recorded;
Ult. load.	34 110	0.3560	+	0.035	= 0.3910	column failed.

* *Transactions*, Am. Soc. C. E., Vol. xiii.

Test No. 206.—T-bar; 2½ ins. × 2½ ins. × 82.375 ins. long over all, 2-in. balls and plates, 80.375 ins. effective length.

$$I = a\,r^2 = 1.73 \text{ sq. ins.} \times (0.55)^2 = 0.523325 \text{ in.}^4$$

Estimated values: $E = 32\,000\,000$ lbs., $\varepsilon = 0.035$ in., $V = +\,0.05$ in.

P = load, in pounds.	(Δ Inches.	+	V) Inches.	= D calculated. Inches.	D observed. Inches.
500 ..	0.00086	+	0.05	= 0.0508.......	0.03
3 000 ...	0.0058	+	0.05	= 0.0558.......	0.05
6 000....	0.0133	+	0.05	= 0.0633.......	0.06
9 000....	0.0238	+	0.05	= 0.0738.......	0.07
12 000....	0.0392	+	0.05	= 0.0892.......	0.10
15 000....	0.0636	+	0.05	= 0.1136.......	0.12
18 000....	0.1097	+	0.05	= 0.1597.......	0.16
20 000....	0.1718	+	0.05	= 0.2218.......	0.22
21 000....	0.2270	+	0.05	= 0.2270.......	0.30
Ult. 21 500....	0.2660	+	0.05	= 0.3160.......	$\propto$ failed.
(22 000)...	(0.3200	+	0.05)	= (0.3700).......	....
(23 000)...	(0.5120	+	0.05)	= (0.5620).......	....

Test No. 208.—T-bar; 1½ ins. × 1½ ins. × 81.1875 ins. long over all, on 1-in. balls and plates, therefore, effective length = 80.1875 ins.

$$I = a\,r^2 = 0.53 \text{ sq. in.} \times (0.32)^2 = 0.054272 \text{ in.}^4.$$

Estimated values: $E = 30\,000\,000$ lbs., $\varepsilon = 0.05$ in., $V = 0.03$ in.

P = load, in pounds.	(Δ Inches.	+	V) Inches.	= D calculated. Inches.	D observed. Inches.
200....	0.00538	+	0.03	= 0.0354......	0.02
600....	0.01965	+	0.03	= 0.0497......	0.05
800....	0.29380	+	0.03	= 0.0594......	0.06
1 000....	0.04188	+	0.03	= 0.0719......	0.07
1 200....	0.05840	+	0.03	= 0.0884......	0.09
1 400. ..	0.08135	+	0.03	= 0.1114......	0.12
1 600....	0.11525	+	0.03	= 0.1453......	0.15
1 800....	0.17100	+	0.03	= 0.2010......	0.20
2 000....	0.27750	+	0.03	= 0.3075......	0.30
Ult. 2 200....	0.51600	+	0.03	= 0.5460......	$\propto$ failed.
(2 250)...	(0.74000	+	0.03)	= (0.7700)......	....
(2 300)...	(1.04200	+	0.03)	= (1.0720)......	....

Test No. 211.—⊤-bar; 2 ins. × 2 ins. x 63.1875 ins. long over all, on 1-in. balls and plates, therefore, effective length = 62.1875 ins.

$$I = a r^2 = (0.95 \text{ sq. in.}) \times (0.43)^2 = 0.175655 \text{ in.}^4$$

Estimated values: $E = 27\ 000\ 000$ lbs., $\varepsilon = 0.10$ in., $V = 0.02$ in.

	P = load, in pounds.	(Δ Inches.	+ V) Inches.	= D calculated. Inches.	D observed. Inches.
	500	0.00533	+ 0.02	= 0.0253	0.03
	2 000	0.02455	+ 0.02	= 0.0446	0.05
	3 000	0.04110	+ 0.02	= 0.0611	0.06
	4 000	0.06175	+ 0.02	= 0.0818	0.08
	5 000	0.08860	+ 0 02	= 0.1086	0.10
	6 000	0.12460	+ 0.02	= 0.1446	0.15
	7 000	0.17600	+ 0.02	= 0.1960	0.20
	8 000	0.25450	+ 0.02	= 0.2745	0.27
	9 000	0.38900	+ 0.02	= 0.4090	0.40
	9 500	0.50100	+ 0.02	= 0.5210	not recorded.
Ult. load.	9 510	0.50400	+ 0.02	= 0.5240	$\propto$ failed.
	9 550	0.51600	+ 0.02	= 0.5360	....

Test No. 217.—⊤-bar; 1 in. × 1 in. × 45¼ ins., long over all, on 1-in. balls and plates, therefore, effective length = 44.25 ins.

$$I = a r^2 = 0.3 \text{ sq. in.} \times (0.26)^2 = 0.02028 \text{ in.}^4$$

Estimated values: $E = 18\ 500\ 000$ lbs., $\varepsilon = 0.037$ in., $V = 0.008$ in.

	P = load, in pounds.	(Δ Inches.	+ V) Inches.	= D calculated. Inches.	D observed. Inches.
	100	0.0025	+ 0.008	= 0.0106	0.01
	400	0.0123	+ 0.008	= 0.0203	0.02
	600	0.0215	+ 0.008	= 0.0295	0.03
	800	0.0342	+ 0 008	= 0.0422	0 04
	1 000	0.0533	+ 0.008	= 0.0613	0.06
	1 200	0.0832	+ 0.008	= 0 0912	0.10
	1 400	0.1415	+ 0.008	= 0.1495	0 15
	1 600	0.2968	+ 0.008	= 0.3048	0.29
	1 700	0.5410	+ 0.008	= 0.5490	0 55
Ult. load.	1 750	0.8675	+ 0.008	= 0.8755	$\propto$ failed.
	1 800	2.0250	+ 0.008	= 2.0330	....

With regard to the value of the modulus of elasticity E, as estimated for the foregoing comparative examples, it may be mentioned that Mr. Hodgkinson found from transverse bending tests of the Low

Moor No. 3 cast iron, that E ranged from 13 585 530 to 14 251 950 lbs., and Mr. James Christie* found from bending tests of the wrought iron upon which he experimented, that E ranged from 19 164 000 to 33 631 000 lbs.

The value of E, as found by bending tests, is necessarily that to which we must refer in dealing with column strength and stiffness, as the modulus of elasticity only enters into the column formula on account of the bending moments exerted on the column, and not at all in connection with direct compressive stresses.

It may be noted here that the value of E obtained from the transverse bending tests on ordinary cold-straightened wrought-iron or steel bars will depend upon the position of the points at which the straightening press has been applied. If the straightening is done near the center of the span, the value of E may reasonably be expected to come out very low, while the influence of any straightening done near the ends of the bars will have comparatively little influence on the results obtained.

In direct tensile tests the position of the points of straightening will have no influence on the results, which will only be affected by the amount of straightening to which the bar has been subjected. This will explain the much greater uniformity in the results obtained by direct tension, as compared with those obtained by transverse bending tests, and also with those obtained from compression tests where the slightest latitude is given for the specimen to act as a column, and where the material has had to be cold-straightened.†

General Formulas for the Relation of Column Proportions to Column Strength.

Round or pivoted ends—

$$\text{Formula (7)}\ R = \frac{l}{r} = \sqrt{\frac{48\,E}{5\,F_c + f_d\left(\frac{c\,\varepsilon}{r^2} - 5\right)}\left[\frac{F_c}{f_d} - 1 - \frac{c\,\varepsilon}{r^2}\right]}$$

for failure by compressive stress,

$$\text{or (8)}\ R = \frac{l}{r} = \sqrt{\frac{48\,E}{5\,F_t - f_d\left(\frac{c\,\varepsilon}{r^2} + 5\right)}\left[\frac{F_t}{f_d} - 1 + \frac{c\,\varepsilon}{r^2}\right]}$$

for failure by tensile stress.

* "The Strength and Elasticity of Structural Steel," *Transactions*, Am. Soc. C. E., Vol. xiii.

† See results of Mr. Christie's tests in "The Strength and Elasticity of Structural Steel," *Transactions*, Am. Soc. C. E., Vol. xiii.

Fixed ends.... R = twice the values given by Formulas (7) and (8).

Flat ends..... R = same as fixed ends, keeping in view that in Formula (8) F_t is to be made zero, resulting in Formula (9).

Hinged or Pin ends. } .. R = { in upper limits, same as for fixed ends. in lower limits, same as for round ends.

Attention must here be drawn to the fact that Formulas (7) and (8), are not actually two different formulas, but are only simple algebraic transformations of one and the same general formula (6), referred to the two conditions of failure by compressive stress and by tensile stress, the only other modifications necessary for their application to any of the types of column previously given being due to the conditions of end fixing as determining the relative length of columns of the same strength, but with different end conditions.

It will be noticed that there are three factors in the formulas to which it is necessary to assign values, and these are:

(1) The value of E = modulus of elasticity;

(2) The value of F_c or F_t = the maximum fiber stress;

(3) The value of $\frac{c\,\varepsilon}{r^2}$.

Careful study of the great variations shown in actual tests, and of the comparisons made between actual and calculated deflections, apparently indicate very great difficulty in assigning any fixed values for these quantities for general application, but it must be remembered, that it is not practically possible to predict the precise strength of any given column, and that being so, it only remains to endeavor to determine the limits between which we may expect the column strength to lie. With this in view, using approximately normal values of E as obtained by tests in direct tension or compression, assigning values to F_c approximately as shown by the higher tests of short columns, and to F_t as given by direct tests of tensile strength, the writer has found that the lower limit of column strength is given fairly by the formulas when $\frac{c\,\varepsilon}{r^2} = 0.6$; and it also appears that the upper limit given by the formulas, when $\frac{c\,\varepsilon}{r^2} = 0.15$ is rarely exceeded.

These values are applicable alike to cast iron, wrought iron, mild steel, hard steel, and several kinds of timber, as will be seen on reference to the various diagrams of column tests appended.

It is interesting to note that in the case of solid, round columns, the value $\frac{c\,\varepsilon}{r^2} = 0.6$ for lower limit strength corresponds to an equivalent eccentricity of (0.3 × radius of gyration), and applying this to a bar of 1 in. diameter, where $c = 0.5$ in., and $r = 0.25$ in., the value of ε would only be (0.3 × 0.25 in.) = 0.075 in.

Of this amount the possible errors in setting the test specimen may be only a small part, and if we assume that initial external curvature of bar, irregularities in the line of physical axis, and the effects of cold-straightening account for say 60% of the total value of ε, the remaining 40%, or 0.03 in., would be the extreme permissible amount of error in setting the specimen, and it becomes apparent how important such small errors are in experiments on columns, and how very carefully the testing must be carried out in order to develop even the lower limit strength shown by the accompanying diagrams.

This also impresses the mind with the danger of generalizing from the results of any single series of tests where the number of tests is not very large.

Strictly speaking, the theory and resulting formulas of this paper apply only when the loads are such as will not stress the material of the column beyond the elastic limit, a condition which applies equally to the common theory of flexure of solid beams under simple bending. This, however, forms no bar to the extension of the application of the formula for elastic beam strength to cases of ultimate strength where the elastic limit is exceeded, provided proper recognition is given to the fact that the ultimate maximum fiber stresses apparently developed are not true values of tensile or compressive strength, but are always much higher than obtained from tests under direct stress. The reasons for this are now well known and need not be dealt with here.

There is, therefore, considerable justification for the application of the column formulas to a comparison with experiments on ultimate strength, which, in fact, form the only available basis of reference to which the engineer can appeal.

Differences of opinion exist among engineers as to whether columns should be designed with regard to the ultimate strength or with regard to the maximum fiber stress developed by the working load. The former is by far the more common practice, owing to the existing state of knowledge as to the principles of column strength, and this renders

it necessary for any theory and formulas, proposed for use in practice, to be compared with available experimental evidence.

In the writer's view, the more rational method is to design columns so as to ensure that given maximum fiber stresses will not be exceeded under the working load, while at the same time taking care to refer to experimental results, in order to see that a sufficient margin is provided against failure by instability in the longer lengths.

It would be reasonable to expect that the values of F_c and F_t, being maximum fiber stresses, would, when referred to tests carried to ultimate failure, partake somewhat of the nature and value generally accorded to the corresponding maximum apparent fiber stresses determined from tests of ultimate transverse strength of simple solid beams. It is necessary, however, to keep in view that it is hardly possible that they can have such high apparent values as in beam tests, owing to the rapidly accumulating bending moment developed in columns by the increasing deflection, which must evidently be intensified by any extra yielding of the extreme fibers due to their elastic limit being exceeded, and thus accelerating the deflection.

In the case of beams, of course, the increase in deflection accompanying an increasing load has no influence whatever on the bending moment.

The writer has not found it necessary for "centrally" loaded columns of wrought iron, mild or hard steel, and timber, to deal with the condition of failure by tensile stress, excepting as regards incipient tension in flat-ended columns.

The maximum compressive stress in a column of symmetrical section always has a greater value in pounds per square inch than the maximum tensile stress, and the difference between the compressive and tensile strengths must be considerable before tension becomes the controlling influence. This is the case with high-class cast iron, such as was used by Hodgkinson in his tests of Low Moor, No. 3 iron, and the curves for failure by tension are therefore plotted on the diagrams showing these tests.

In order to discover, if possible, any special features accompanying any particular set of tests, the writer has assigned a separate diagram to the principal sets of experiments where the number and range of tests were sufficiently great to justify it, and where this was not the case, the individual tests have been given distinguishing symbols to assist in arriving at a correct judgment.

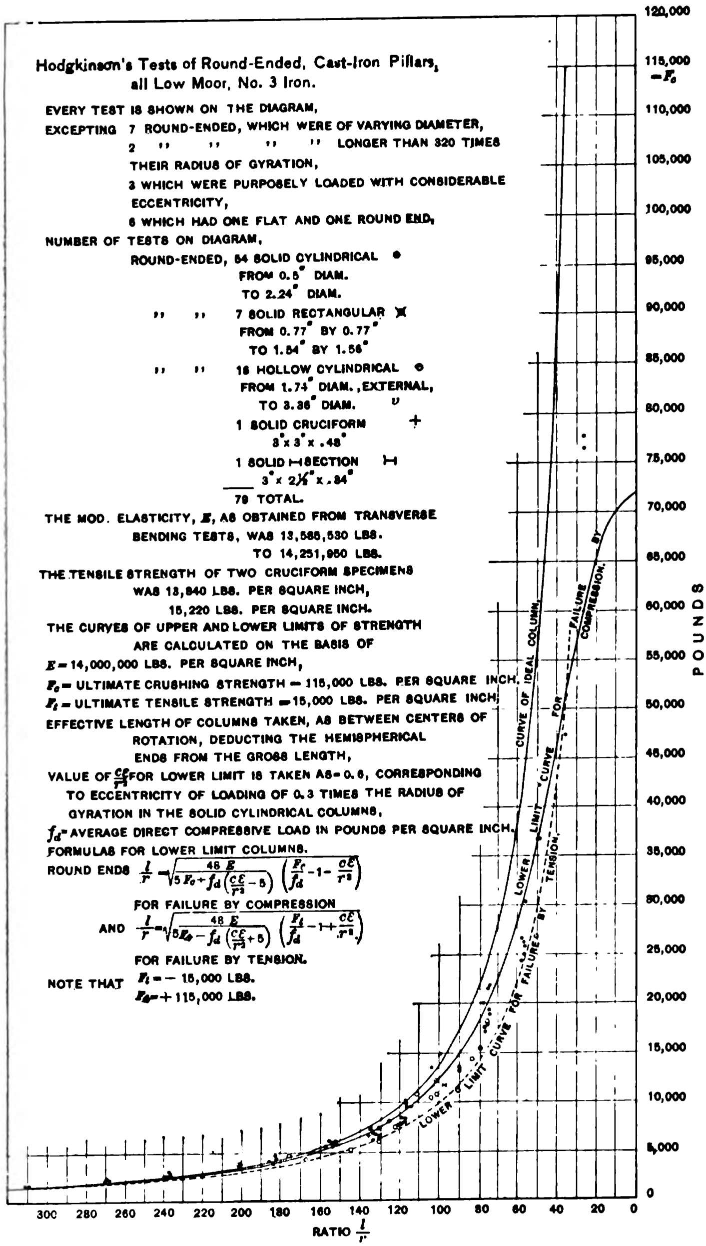

Hodgkinson's Tests of Round-Ended, Cast-Iron Pillars,
all Low Moor, No. 3 Iron.
EVERY TEST IS SHOWN ON THE DIAGRAM,
EXCEPTING 7 ROUND-ENDED, WHICH WERE OF VARYING DIAMETER,
2 " " " " LONGER THAN 320 TIMES THEIR RADIUS OF GYRATION,
3 WHICH WERE PURPOSELY LOADED WITH CONSIDERABLE ECCENTRICITY,
6 WHICH HAD ONE FLAT AND ONE ROUND END,
NUMBER OF TESTS ON DIAGRAM,
ROUND-ENDED, 54 SOLID CYLINDRICAL FROM 0.5" DIAM. TO 2.24" DIAM.
" " 7 SOLID RECTANGULAR FROM 0.77" BY 0.77" TO 1.54" BY 1.56"
" " 15 HOLLOW CYLINDRICAL FROM 1.74" DIAM., EXTERNAL, TO 3.36" DIAM.
1 SOLID CRUCIFORM 3" x 3" x .48"
1 SOLID H-SECTION 3" x 2½" x .34"
79 TOTAL.
THE MOD. ELASTICITY, E, AS OBTAINED FROM TRANSVERSE BENDING TESTS, WAS 13,585,530 LBS. TO 14,251,950 LBS.
THE TENSILE STRENGTH OF TWO CRUCIFORM SPECIMENS WAS 13,840 LBS. PER SQUARE INCH, 15,220 LBS. PER SQUARE INCH.
THE CURVES OF UPPER AND LOWER LIMITS OF STRENGTH ARE CALCULATED ON THE BASIS OF
E = 14,000,000 LBS. PER SQUARE INCH,
F_c = ULTIMATE CRUSHING STRENGTH = 115,000 LBS. PER SQUARE INCH,
F_t = ULTIMATE TENSILE STRENGTH = 15,000 LBS. PER SQUARE INCH,
EFFECTIVE LENGTH OF COLUMNS TAKEN, AS BETWEEN CENTERS OF ROTATION, DEDUCTING THE HEMISPHERICAL ENDS FROM THE GROSS LENGTH,
VALUE OF $\frac{c\varepsilon}{r^2}$ FOR LOWER LIMIT IS TAKEN AS = 0.6, CORRESPONDING TO ECCENTRICITY OF LOADING OF 0.3 TIMES THE RADIUS OF GYRATION IN THE SOLID CYLINDRICAL COLUMNS,
f_d = AVERAGE DIRECT COMPRESSIVE LOAD IN POUNDS PER SQUARE INCH.
FORMULAS FOR LOWER LIMIT COLUMNS.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48\,E}{5F_c + f_d\left(\frac{c\varepsilon}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{c\varepsilon}{r^2}\right)}$
FOR FAILURE BY COMPRESSION
AND $\frac{l}{r} = \sqrt{\frac{48\,E}{5F_t - f_d\left(\frac{c\varepsilon}{r^2} + 5\right)}\left(\frac{F_t}{f_d} - 1 + \frac{c\varepsilon}{r^2}\right)}$
FOR FAILURE BY TENSION.
NOTE THAT F_t = − 15,000 LBS.
F_c = + 115,000 LBS.
CURVE OF IDEAL COLUMN.
FAILURE BY COMPRESSION.
LOWER LIMIT CURVE FOR FAILURE BY TENSION.
LOWER LIMIT CURVE FOR FAILURE BY TENSION.
120,000
115,000 = F_c
110,000
105,000
100,000
95,000
90,000
85,000
80,000
75,000
70,000
65,000
60,000
55,000
50,000
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
POUNDS
300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$

FIG. 17.

As has already been remarked, the writer has been unable to obtain practical verification of the precise influence of form of section from existing records of experiments. To demonstrate this influence by practical evidence it would be necessary to make a large number of new experiments on sections having widely different values of $\frac{c}{r}$, and at the same time to impose the loads with a comparatively large and predetermined value of eccentricity, in order to overshadow the relative influence of what has been dealt with in the foregoing pages as "equivalent eccentricity."

All the diagrams of column tests accompanying this paper have been made self-explanatory as far as possible.

They are as follow:

Cast Iron.

Fig. 17.—Representing 79 tests of round-ended columns of Low Moor. No. 3 cast iron, by Mr. Hodgkinson.

The tests are plotted to effective lengths, an allowance having been made for the rounding of the ends by taking them as hemispherical of the same diameter as the bars. The ends were not actually hemispherical in every case, some being somewhat more pointed, but no error of importance is involved in the assumption made.

Fig. 18. Representing 96 tests of flat-ended and disc-ended columns of Low Moor, No. 3 cast iron, by Mr. Hodgkinson.

It will be noticed in this set of tests on flat-ended columns, that the longer columns, from a ratio of 80 upward, do not show so low a strength as is indicated by the lowermost dotted curve for incipient tension, but failure at this critical point in flat-ended columns is evidenced so strongly in the case of tests of wrought iron and steel that the higher results obtained by Hodgkinson in these tests on cast iron must be attributed partly to his extreme care, and partly to the comparative fewness of tests of each length.

These two sets of tests have been abstracted from Mr. Hodgkinson's remarkably careful records.* They are the only published records of tests of cast-iron columns made in a consistent and scientific manner on one grade of material of known character. The values of load per square inch, radius of gyration and ratio $\frac{l}{r}$, have been calculated by the writer from Mr. Hodgkinson's figures.

* *Philosophical Transactions*, Royal Society of London, 1840.

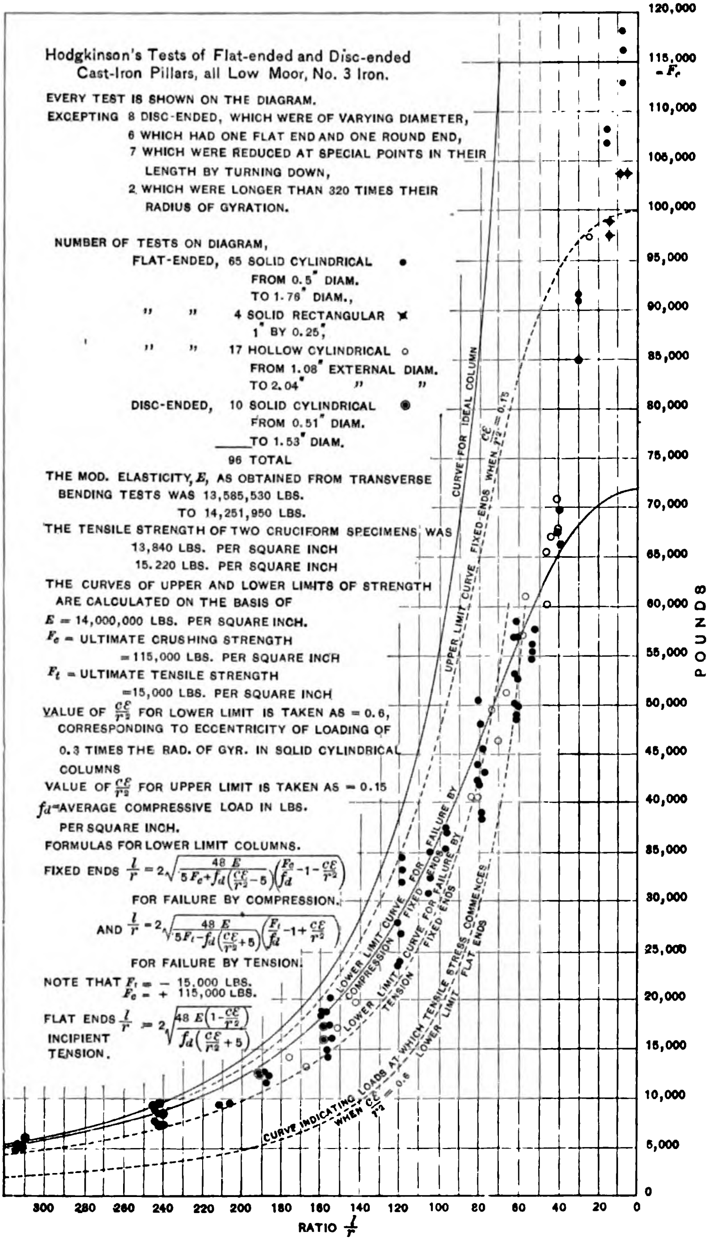

Hodgkinson's Tests of Flat-ended and Disc-ended Cast-Iron Pillars, all Low Moor, No. 3 Iron.
EVERY TEST IS SHOWN ON THE DIAGRAM.
EXCEPTING 8 DISC-ENDED, WHICH WERE OF VARYING DIAMETER,
6 WHICH HAD ONE FLAT END AND ONE ROUND END,
7 WHICH WERE REDUCED AT SPECIAL POINTS IN THEIR LENGTH BY TURNING DOWN,
2 WHICH WERE LONGER THAN 320 TIMES THEIR RADIUS OF GYRATION.
NUMBER OF TESTS ON DIAGRAM,
FLAT-ENDED, 65 SOLID CYLINDRICAL FROM 0.5" DIAM. TO 1.76" DIAM.,
" " 4 SOLID RECTANGULAR 1" BY 0.25",
" " 17 HOLLOW CYLINDRICAL FROM 1.08" EXTERNAL DIAM. TO 2.04" " "
DISC-ENDED, 10 SOLID CYLINDRICAL FROM 0.51" DIAM. TO 1.53" DIAM.
96 TOTAL
THE MOD. ELASTICITY, E, AS OBTAINED FROM TRANSVERSE BENDING TESTS WAS 13,585,530 LBS. TO 14,251,950 LBS.
THE TENSILE STRENGTH OF TWO CRUCIFORM SPECIMENS WAS 13,840 LBS. PER SQUARE INCH 15.220 LBS. PER SQUARE INCH
THE CURVES OF UPPER AND LOWER LIMITS OF STRENGTH ARE CALCULATED ON THE BASIS OF
E = 14,000,000 LBS. PER SQUARE INCH.
F_c = ULTIMATE CRUSHING STRENGTH = 115,000 LBS. PER SQUARE INCH
F_t = ULTIMATE TENSILE STRENGTH = 15,000 LBS. PER SQUARE INCH
VALUE OF $\frac{c\varepsilon}{r^2}$ FOR LOWER LIMIT IS TAKEN AS = 0.6, CORRESPONDING TO ECCENTRICITY OF LOADING OF 0.3 TIMES THE RAD. OF GYR. IN SOLID CYLINDRICAL COLUMNS
VALUE OF $\frac{c\varepsilon}{r^2}$ FOR UPPER LIMIT IS TAKEN AS = 0.15
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
FORMULAS FOR LOWER LIMIT COLUMNS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E}{5F_c+f_d\left(\frac{c\varepsilon}{r^2}-5\right)}\left(\frac{F_c}{f_d}-1-\frac{c\varepsilon}{r^2}\right)}$
FOR FAILURE BY COMPRESSION.
AND $\frac{l}{r} = 2\sqrt{\frac{48\,E}{5F_t-f_d\left(\frac{c\varepsilon}{r^2}+5\right)}\left(\frac{F_t}{f_d}-1+\frac{c\varepsilon}{r^2}\right)}$
FOR FAILURE BY TENSION.
NOTE THAT F_t = − 15,000 LBS. F_c = + 115,000 LBS.
FLAT ENDS INCIPIENT TENSION. $\frac{l}{r} = 2\sqrt{\frac{48\,E\left(1-\frac{c\varepsilon}{r^2}\right)}{f_d\left(\frac{c\varepsilon}{r^2}+5\right)}}$
CURVE FOR IDEAL COLUMN
UPPER LIMIT CURVE FIXED ENDS WHEN $\frac{c\varepsilon}{r^2}$ = 0.15
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION FIXED ENDS
LOWER LIMIT CURVE FOR FAILURE BY TENSION FIXED ENDS
CURVE INDICATING LOADS AT WHICH TENSILE STRESS COMMENCES WHEN $\frac{c\varepsilon}{r^2}$ = 0.6 LOWER LIMIT FLAT ENDS
120,000
115,000 = F_c
110,000
105,000
100,000
95,000
90,000
85,000
80,000
75,000
70,000
65,000
60,000
55,000
50,000
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
POUNDS
300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$

FIG. 18.

Fig. 19.—Representing 76 tests of cylindrical, flat-ended columns of cast iron of various kinds, by Mr. Hodgkinson.*

One test by Mr. Charnock, at Bradford Technical College, England, on a hollow cast-iron column, with flat flanged ends.†

One test by Professor John Goodman, at Yorkshire College, Leeds, England, on a hollow column with flanged ends.‡

Fourteen tests on cylindrical, hollow columns, with flat ends, made by the New York City Department of Buildings.§

Fourteen tests on hollow columns with flat and flanged ends, at Watertown Arsenal.||

Total number of tests on diagram, 106.

The sectional areas, loads per square inch, radii of gyration and values of $\frac{l}{r}$, have all been calculated by the writer from the figures given in the records, except for the New York tests, for which the radii of gyration and ratio $\frac{l}{r}$, only, were calculated.

Most of the Watertown tests were on tapered columns, and all the areas, loads per square inch and radii of gyration calculated for these refer to the section at the middle of the column length, which will explain the divergence from the figures for ultimate strength per square inch given by Professor Lanza.

The results plotted on Fig. 19 refer to tests of columns of various kinds of cast iron. Hodgkinson's tests alone cover 17 different irons, of widely different compressive strength, in short specimens. No information is given as to the physical characteristics of the irons used in the other tests plotted on the diagram.

A number of the columns of this (1857) series by Hodgkinson were subjected to more than one test. They were made long at first, and after being tested they were cut down into shorter columns and re-tested, a circumstance still further adding to the difficulty of deriving any definite laws of strength from these tests.

Regarded as a means of determining the influence of column proportions on ultimate strength, the results of any of the tests plotted on Fig. 19 are of little value, and a most cursory consideration will

*Abstracted from Mr. Hodgkinson's paper in *Philosophical Transactions*, Royal Society of London, 1857.

† *Engineering*, February 28th, 1896.

‡ *Engineering*, September 11th, 1896.

§ *Engineering News*, January 13th, 1898.

|| Reports, 1887-1888, and Lanza's "Applied Mechanics."

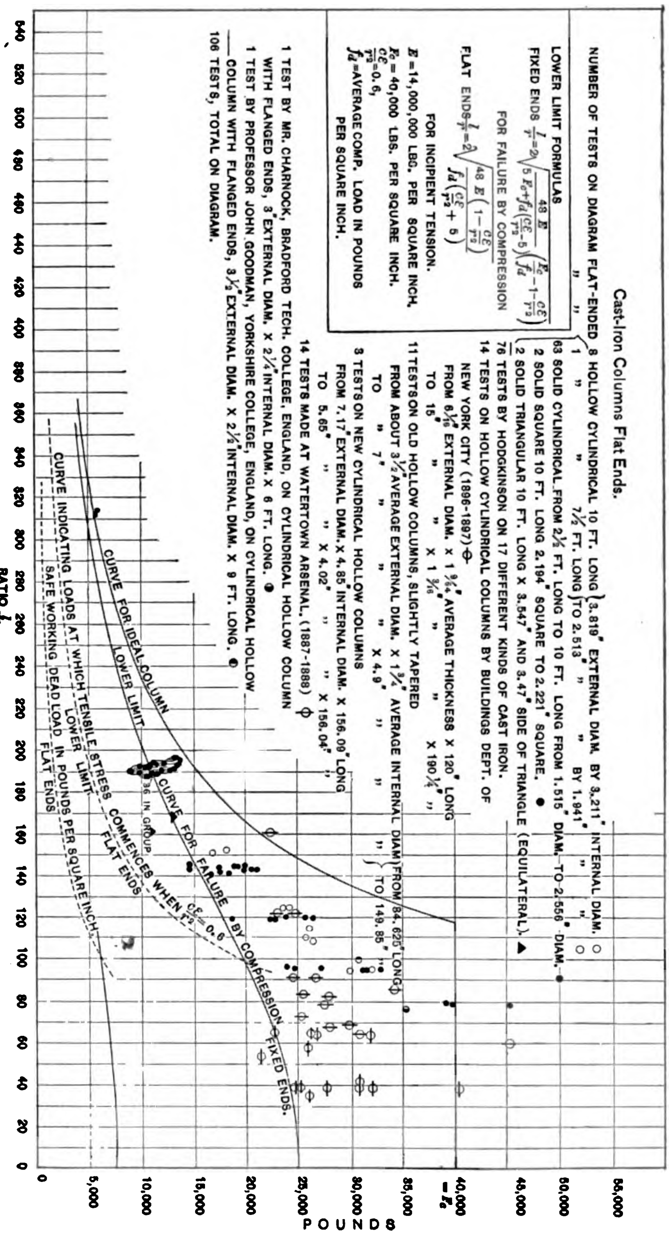

Cast-Iron Columns Flat Ends.
NUMBER OF TESTS ON DIAGRAM FLAT-ENDED 8 HOLLOW CYLINDRICAL 10 FT. LONG 3.819" EXTERNAL DIAM. BY 3.211" INTERNAL DIAM. ○
" " 1 " " 7½ FT. LONG TO 2.513" " " BY 1.941" " " ○
63 SOLID CYLINDRICAL FROM 2½ FT. LONG TO 10 FT. LONG FROM 1.515" DIAM. TO 2.556" DIAM. ●
2 SOLID SQUARE 10 FT. LONG 2.194" SQUARE TO 2.221" SQUARE. ●
2 SOLID TRIANGULAR 10 FT. LONG X 3.547" AND 3.47" SIDE OF TRIANGLE (EQUILATERAL). ▲
76 TESTS BY HODGKINSON ON 17 DIFFERENT KINDS OF CAST IRON.
14 TESTS ON HOLLOW CYLINDRICAL COLUMNS BY BUILDINGS DEPT. OF NEW YORK CITY (1896-1897)
FROM 6 1/16" EXTERNAL DIAM. X 1 9/64" AVERAGE THICKNESS X 120" LONG
TO 15" " " X 1 3/16" " " X 190¼" "
11 TESTS ON OLD HOLLOW COLUMNS, SLIGHTLY TAPERED
FROM ABOUT 3½" AVERAGE EXTERNAL DIAM. X 1¾" AVERAGE INTERNAL DIAM. FROM 84.625" LONG
TO " 7" " " " X 4.9" " " " TO 149.85" "
3 TESTS ON NEW CYLINDRICAL HOLLOW COLUMNS
FROM 7.17" EXTERNAL DIAM. X 4.85" INTERNAL DIAM. X 156.09" LONG
TO 5.65" " " X 4.02" " " X 156.04" "
14 TESTS MADE AT WATERTOWN ARSENAL. (1887-1888)
1 TEST BY MR. CHARNOCK, BRADFORD TECH. COLLEGE, ENGLAND, ON CYLINDRICAL HOLLOW COLUMN WITH FLANGED ENDS, 3" EXTERNAL DIAM. X 2¼" INTERNAL DIAM. X 6 FT. LONG.
1 TEST BY PROFESSOR JOHN GOODMAN, YORKSHIRE COLLEGE, ENGLAND, ON CYLINDRICAL HOLLOW COLUMN WITH FLANGED ENDS, 3½" EXTERNAL DIAM. X 2½" INTERNAL DIAM. X 9 FT. LONG.
106 TESTS, TOTAL ON DIAGRAM.
LOWER LIMIT FORMULAS
FIXED ENDS $\frac{l}{r}=2\sqrt{\frac{48\ E}{5\ F_c+f_d\left(\frac{c\mathcal{E}}{r^2}-5\right)}\left(\frac{F_c}{f_d}-1-\frac{c\mathcal{E}}{r^2}\right)}$
FOR FAILURE BY COMPRESSION
FLAT ENDS $\frac{l}{r}=2\sqrt{\frac{48\ E\left(1-\frac{c\mathcal{E}}{r^2}\right)}{f_d\left(\frac{c\mathcal{E}}{r^2}+5\right)}}$
FOR INCIPIENT TENSION.
E = 14,000,000 LBS. PER SQUARE INCH.
F_c = 40,000 LBS. PER SQUARE INCH.
$\frac{c\mathcal{E}}{r^2}$ = 0.6,
f_d = AVERAGE COMP. LOAD IN POUNDS PER SQUARE INCH.
CURVE FOR IDEAL COLUMN
LOWER LIMIT
CURVE FOR FAILURE BY COMPRESSION
FIXED ENDS.
36 IN GROUP
CURVE INDICATING LOADS AT WHICH TENSILE STRESS COMMENCES WHEN $\frac{c\mathcal{E}}{r^2}$ = 0.6
LOWER LIMIT FLAT ENDS
SAFE WORKING DEAD LOAD IN POUNDS PER SQUARE INCH.
FLAT ENDS
540 520 500 480 460 440 420 400 380 360 340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 $=F_c$ 45,000 50,000 55,000
POUNDS

FIG. 19.

show the absurdity of attempting to generalize with regard to any of the three principal sets of tests of this diagram. Nevertheless, these are the most important tests of cast-iron columns yet made, from the point of view of the engineer, representing, as they do, all the experimental evidence at present available to justify the confidence of the designer using this material in its commoner qualities without any definite knowledge or check upon its physical characteristics.

It is worthy of note that prior to the Watertown tests of 1887 and 1888, there was absolutely no published experimental evidence existing of the strength of common grade cast-iron columns of the proportions of length to radius of gyration in most common use. Hodgkinson's tests (1857) did not give results on columns of shorter lengths than 79 or 80 times the radius of gyration, and his tests in the 1840 paper were on iron of a comparatively high class. This want has, to a slight extent, been filled by the Watertown tests and the New York Building Department tests.

It is surprising to think of the enormous number of cast-iron columns which have been put into use without any justification for the loads imposed on them, except a simple faith in Hodgkinson's, Gordon's and Rankine's formulas, and in the numerous tables calculated therefrom and published in engineering pocketbooks and treatises.

The writer does not pretend that the curves plotted on Fig. 19 have any other than a purely accidental correspondence with the experimental results shown on the diagram.

The curves calculated from the formula refer to material of certain fixed characteristics, while the experiments plotted on Fig. 19 were made on cast iron of widely different grades.

At the same time the curves follow definite laws, and may serve as a basis of reference. The writer, in his own practice, would not care to count upon higher ultimate strengths for common cast-iron columns than are given by the lowest curves on the diagram.

Wrought Iron.—Round or Pivoted Ends.

Fig. 20.—Representing 33 tests of round-ended columns by Mr. Christie.* These tests have all been plotted to lengths measured from center to center of hemispherical ends. Mr. Christie gives values of R based on extreme lengths.

* *Transactions*, Am. Soc. C. E., Vol. xiii.

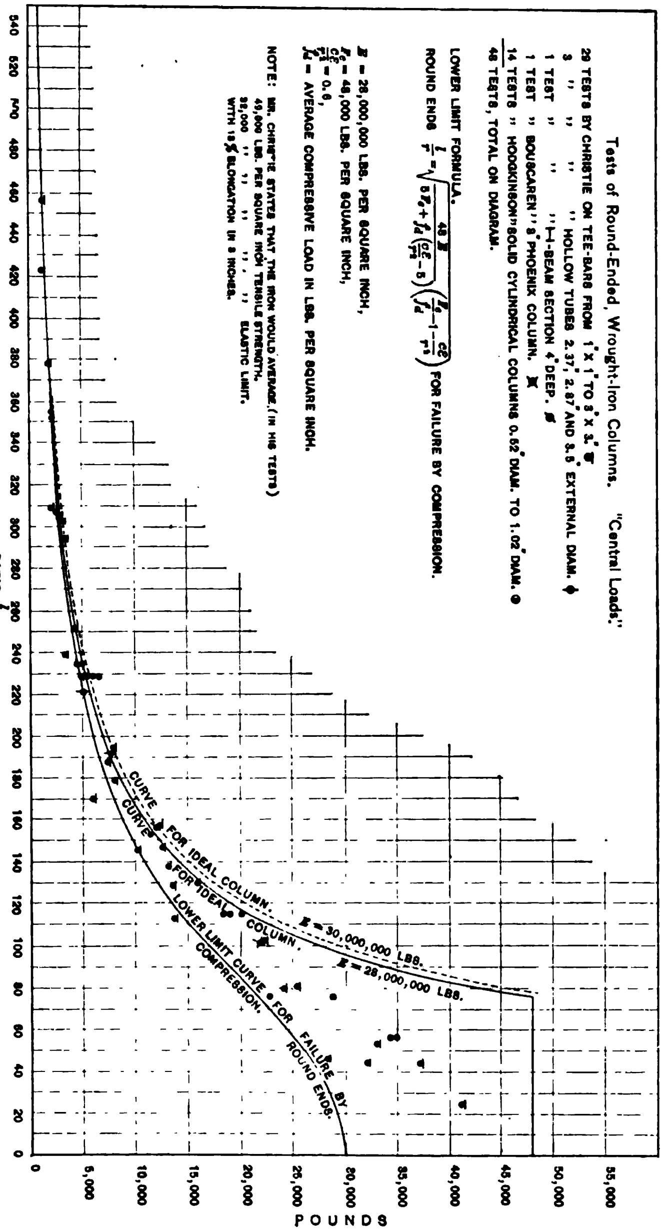

Tests of Round-Ended, Wrought-Iron Columns. "Central Loads."
29 TESTS BY CHRISTIE ON TEE-BARS FROM 1" X 1" TO 3" X 3".
3 " " " HOLLOW TUBES 2.37", 2.87" AND 3.5" EXTERNAL DIAM.
1 TEST " " "I-BEAM SECTION 4" DEEP.
1 TEST " BOUSCAREN'S PHOENIX COLUMN.
14 TESTS " HODGKINSON'S SOLID CYLINDRICAL COLUMNS 0.52" DIAM. TO 1.02" DIAM.
48 TESTS, TOTAL ON DIAGRAM.
LOWER LIMIT FORMULA.
ROUND ENDS FOR FAILURE BY COMPRESSION.
E = 28,000,000 LBS. PER SQUARE INCH,
f_c = 48,000 LBS. PER SQUARE INCH,
f_a = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
NOTE: MR. CHRISTIE STATES THAT THE IRON WOULD AVERAGE (IN HIS TESTS)
49,600 LBS. PER SQUARE INCH TENSILE STRENGTH.
32,000 " " " " ELASTIC LIMIT.
WITH 18% ELONGATION IN 8 INCHES.
CURVE FOR IDEAL COLUMN.
E = 30,000,000 LBS.
CURVE FOR IDEAL COLUMN.
E = 28,000,000 LBS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION.
ROUND ENDS.
RATIO $\frac{l}{r}$
POUNDS

FIG. 30.

In the case of Tests Nos. 227, 228 and 229 the ratio $\frac{l}{r}$ has been recalculated from the lengths given in Mr. Christie's Table No. 6. The values of $\frac{l}{r}$ for these three tests given in this table do not agree with the lengths and radii of gyration.

The 33 tests by Mr. Christie shown on this diagram are Nos. 200 to 229, inclusive, of his Table No. 6, and Nos. 287, 290 and 293 of his Table No. 8.

One test of a round-ended column of large size by L. F. G. Bouscaren,* M. Am. Soc. C. E. The ends of this column were portions of a sphere of 10⅛ ins. radius, and the effective length has, therefore, been taken to be 20¼ ins. shorter than the length over all.

Fourteen tests of round-ended columns by Mr. Hodgkinson.† Allowance for the round ends, in arriving at effective length, has been made in this case also. The sectional areas, loads per square inch, radii of gyration, and values of $\frac{l}{r}$, have been calculated by the writer from Mr. Hodgkinson's records.

Total number of tests on diagram, 48.

Fig. 21.—Representing 116 results, by Professor Tetmajer on pivot-ended columns.‡ These results represent 210 experiments. Ninety-four of the results plotted represent in each case the average of two tests, while twenty-two of the results are for single experiments. The results, as plotted by the writer, are in every case for the load per gross square inch. Some of the specimens were compounded of two or four pieces riveted together, and in the diagram given by Professor J. B. Johnson in his "Materials of Construction," these results appear to have been plotted for load per net square inch (the rivet holes being deducted), and they have, therefore, too high a value. This error was repeated in the reproduction of the diagram by Mr. Marston in his discussion of Professor Cain's paper.§

Fig. 22.—This diagram is simply a combination of Figs. 20 and 21, and therefore includes practically the whole of the available experimental evidence as to the strength of wrought-iron columns with both ends free, and unconstrained, *i. e.*, with "round" or "pivot ends" and with "central" loading.

* *Transactions*, Am. Soc. C. E., Vol. ix.

† *Philosophical Transactions*, Royal Society, London, 1840.

‡ "Tetmajer's Communications."

§ "The Ideal Column," *Transactions*, Am. Soc. C. E., Vol. xxxix, pp. 109 to 111.

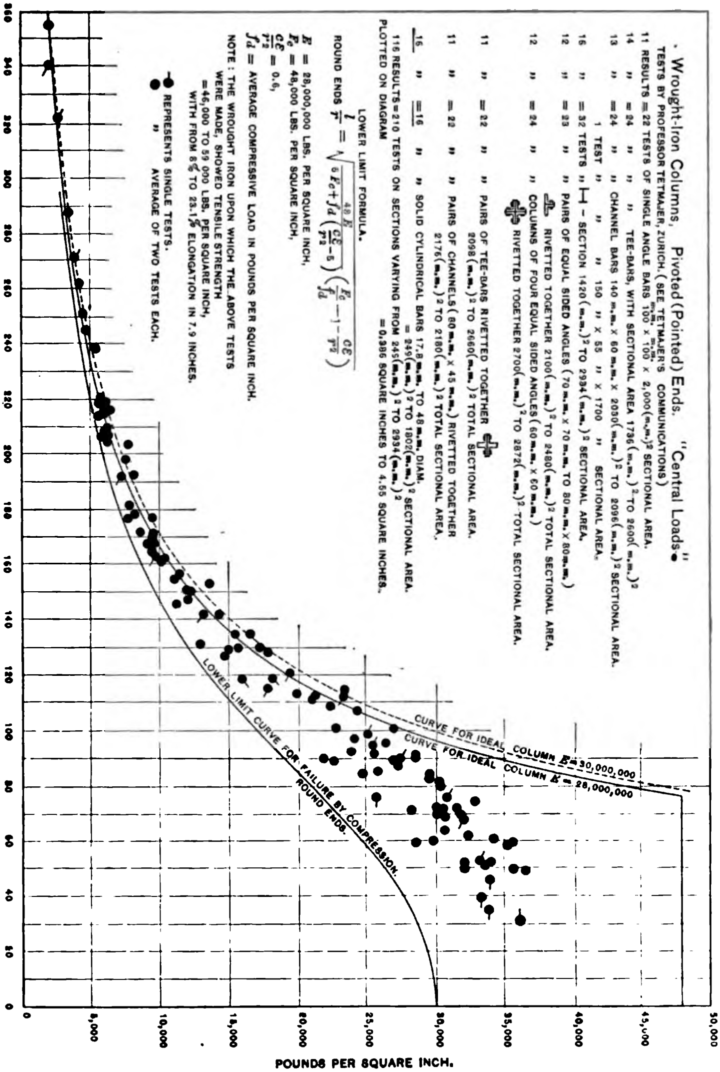

Wrought-Iron Columns, Pivoted (Pointed) Ends. "Central Loads."
TESTS BY PROFESSOR TETMAJER, ZURICH. (SEE TETMAJER'S COMMUNICATIONS)
11 RESULTS = 22 TESTS OF SINGLE ANGLE BARS 100 m.m. x 100 m.m. x 2,000 (m.m.)² SECTIONAL AREA.
14 " = 24 " " " TEE-BARS, WITH SECTIONAL AREA 1736 (m.m.)² TO 2600 (m.m.)²
13 " = 24 " " CHANNEL BARS 140 m.m. x 60 m.m. x 2030 (m.m.)² TO 2096 (m.m.)² SECTIONAL AREA.
1 TEST " " " 150 " x 55 " x 1700 " SECTIONAL AREA.
16 " = 32 TESTS " H - SECTION 1420 (m.m.)² TO 2934 (m.m.)² SECTIONAL AREA.
12 " = 23 " " PAIRS OF EQUAL SIDED ANGLES (70 m.m. x 70 m.m. TO 80 m.m. x 80 m.m.)
RIVETTED TOGETHER 2100 (m.m.)² TO 2480 (m.m.)² TOTAL SECTIONAL AREA.
12 " = 24 " " COLUMNS OF FOUR EQUAL SIDED ANGLES (60 m.m. x 60 m.m.)
RIVETTED TOGETHER 2700 (m.m.)² TO 2872 (m.m.)² TOTAL SECTIONAL AREA.
11 " = 22 " " PAIRS OF TEE-BARS RIVETTED TOGETHER
2098 (m.m.)² TO 2660 (m.m.)² TOTAL SECTIONAL AREA.
11 " = 22 " " PAIRS OF CHANNELS (80 m.m. x 45 m.m.) RIVETTED TOGETHER
2176 (m.m.)² TO 2180 (m.m.)² TOTAL SECTIONAL AREA.
16 " = 16 " " SOLID CYLINDRICAL BARS 17.8 m.m. TO 48 m.m. DIAM.
= 249 (m.m.)² TO 1802 (m.m.)² SECTIONAL AREA.
116 RESULTS = 210 TESTS ON SECTIONS VARYING FROM 245 (m.m.)² TO 2934 (m.m.)²
PLOTTED ON DIAGRAM = 0.386 SQUARE INCHES TO 4.55 SQUARE INCHES.
LOWER LIMIT FORMULA.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48\,E}{6\,F_c + f_d\left(\frac{c\mathcal{E}}{r^2} - 5\right)\left(\frac{F_c}{f_d} - 1 - \frac{c\mathcal{E}}{r^2}\right)}}$
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 LBS. PER SQUARE INCH,
$\frac{c\mathcal{E}}{r^2}$ = 0.6,
f_d = AVERAGE COMPRESSIVE LOAD IN POUNDS PER SQUARE INCH.
NOTE: THE WROUGHT IRON UPON WHICH THE ABOVE TESTS WERE MADE, SHOWED TENSILE STRENGTH = 46,000 TO 59 000 LBS. PER SQUARE INCH, WITH FROM 8% TO 23.1% ELONGATION IN 7.9 INCHES.
REPRESENTS SINGLE TESTS.
" AVERAGE OF TWO TESTS EACH.
CURVE FOR IDEAL COLUMN E = 30,000,000
CURVE FOR IDEAL COLUMN E = 28,000,000
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. ROUND ENDS.
POUNDS PER SQUARE INCH.
0
5,000
10,000
15,000
20,000
25,000
30,000
35,000
40,000
45,000
50,000
RATIO $\frac{l}{r}$
360
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0

FIG. 21.

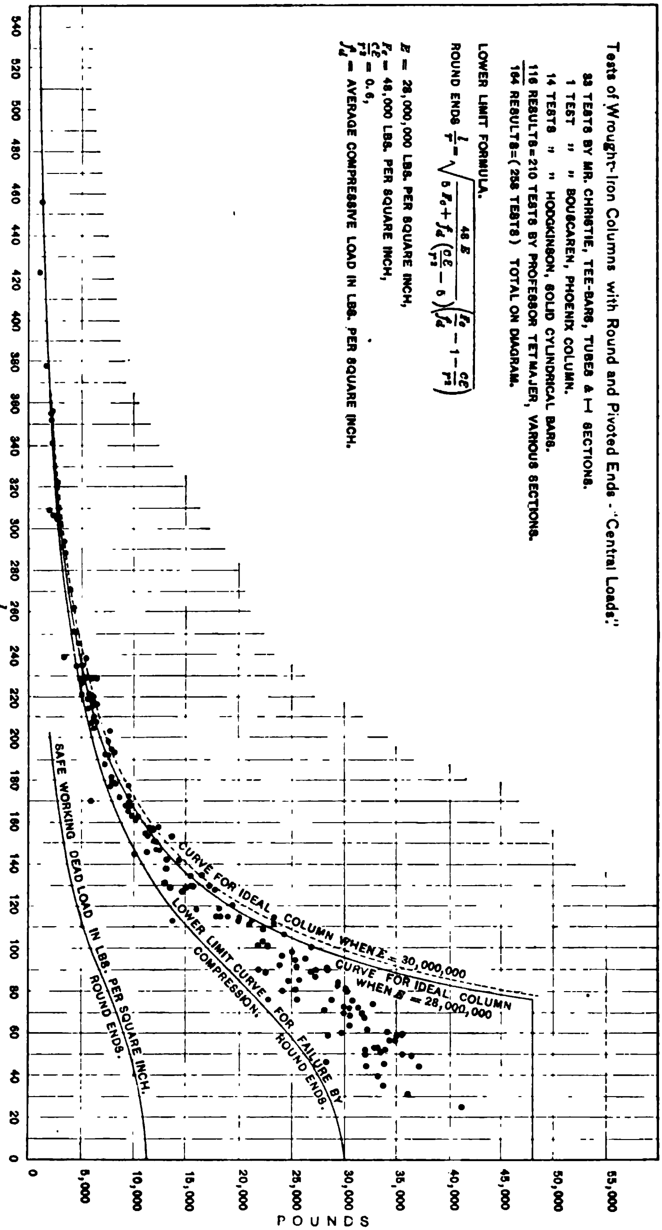
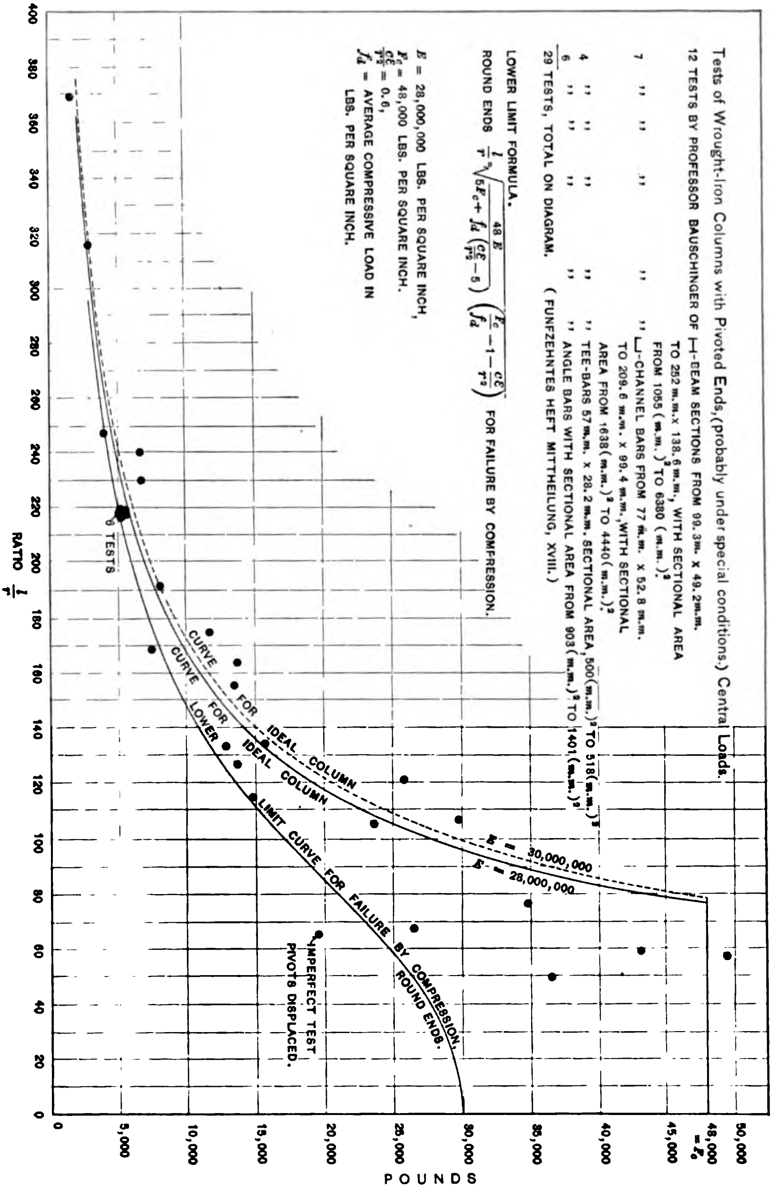

Tests of Wrought-Iron Columns with Round and Pivoted Ends - "Central Loads."
33 TESTS BY MR. CHRISTIE, TEE-BARS, TUBES & H SECTIONS.
1 TEST " " BOUSCAREN, PHOENIX COLUMN.
14 TESTS " " HODGKINSON, SOLID CYLINDRICAL BARS.
116 RESULTS=210 TESTS BY PROFESSOR TETMAJER, VARIOUS SECTIONS.
164 RESULTS=(258 TESTS) TOTAL ON DIAGRAM.
LOWER LIMIT FORMULA.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48\,E}{5\,F_c + f_d\left(\frac{c\epsilon}{r^2} - 6\right)}\left(\frac{F_c}{f_d} - 1 - \frac{c\epsilon}{r^2}\right)}$
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 LBS. PER SQUARE INCH,
$\frac{c\epsilon}{r^2}$ = 0.6,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN WHEN E = 30,000,000
CURVE FOR IDEAL COLUMN WHEN E = 28,000,000
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION, ROUND ENDS.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCH. ROUND ENDS.
RATIO $\frac{l}{r}$
POUNDS
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500 520 540
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000

FIG. 23.



Tests of Wrought-Iron Columns with Pivoted Ends,(probably under special conditions.) Central Loads.
12 TESTS BY PROFESSOR BAUSCHINGER OF I-BEAM SECTIONS FROM 99.3m. X 49.2m.m. TO 252 m.m. X 138.6 m.m., WITH SECTIONAL AREA FROM 1055 (m.m.)² TO 6380 (m.m.)²
7 " " " " ⊔-CHANNEL BARS FROM 77 m.m. X 52.8 m.m. TO 209.6 m.m. X 99.4 m.m., WITH SECTIONAL AREA FROM 1638 (m.m.)² TO 4440 (m.m.)²
4 " " " " TEE-BARS 57 m.m. X 28.2 m.m. SECTIONAL AREA, 500 (m.m.)² TO 518 (m.m.)²
6 " " " " ANGLE BARS WITH SECTIONAL AREA FROM 903 (m.m.)² TO 1401 (m.m.)²
29 TESTS, TOTAL ON DIAGRAM. (FUNFZEHNTES HEFT MITTHEILUNG, XVIII.)
LOWER LIMIT FORMULA.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48\,E}{5F_c + f_d\left(\frac{cE}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{cE}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 LBS. PER SQUARE INCH.
$\frac{cE}{r^2}$ = 0.6,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
6 TESTS
CURVE FOR IDEAL COLUMN
CURVE FOR IDEAL COLUMN
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION, ROUND ENDS.
IMPERFECT TEST PIVOTS DISPLACED.
E = 30,000,000
E = 28,000,000
400 380 360 340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 48,000 $=F_c$ 50,000
POUNDS

FIG 28.

The experiments of M. Considère on pivot-ended columns cannot be made use of as bases of reference for practical work, on account of the attempt made to feel for the "physical axis," or axis of greatest resistance, by moving the pivot ends out of the geometric axis, a refinement which is entirely out of the question in practical construction.

This objection probably applies also to Professor Bauschinger's experiments on pivot-ended columns, but this point the writer has not been able to make out clearly from the records.

It may also be noted that M. Considère's test specimens were of very small sections, the heaviest being only 11 mm. × 23 mm. (rectangular) or 253 sq. mm. = about 0.4 sq. in., and the lightest only 77 sq. mm. (angle section) = about 0.12 sq. in.* This last objection does not apply to Bauschinger's pivot-ended tests, in which the sections ranged from a maximum of 63.8 sq. cm. (⊢⊣-section, 25.2 cm. × 13.86 cm.), or about 9.9 sq. ins. sectional area, to a minimum of 5 sq. cm. (T-bar, 5.7 cm. × 2.82 cm.), or about 0.755 sq. in. sectional area.

Although it has been stated that Bauschinger's tests (pivot-ended) probably cannot be used as bases of reference in practical work, owing to the refinements probably adopted in carrying out the tests, yet it has seemed to the writer that a record of them is necessary to the completeness of this paper; and a still stronger reason for their presentation in diagram form lies in the fact that in spite of all the highly skilled care and accuracy bestowed on the tests, Bauschinger did not succeed in keeping the lower tests above the lower limit found in the tests of other experimenters, as will be seen on reference to Fig. 23.

Fig. 23.—Twenty-nine tests by Professor Bauschinger on pivot-ended columns of wrought iron.†

Fig. 24.—Nine results, representing eighteen experiments by Professor Tetmajer on pivot-ended wrought-iron columns under intentionally eccentric loads. These results are in each case the average of the tests of two specimens.‡ It is unfortunate that the record of each individual experiment has not been given by Professor Tetmajer.

* Considère's Report on "La Resistance au Flambement des Pièces Comprimées" (French Commission des Methodes D'Essai des Materiaux de Construction. Tome iii).

† Fünfzehntes Heft, *Mittheilung*, xviii.

‡ "Tetmajer's Communications."

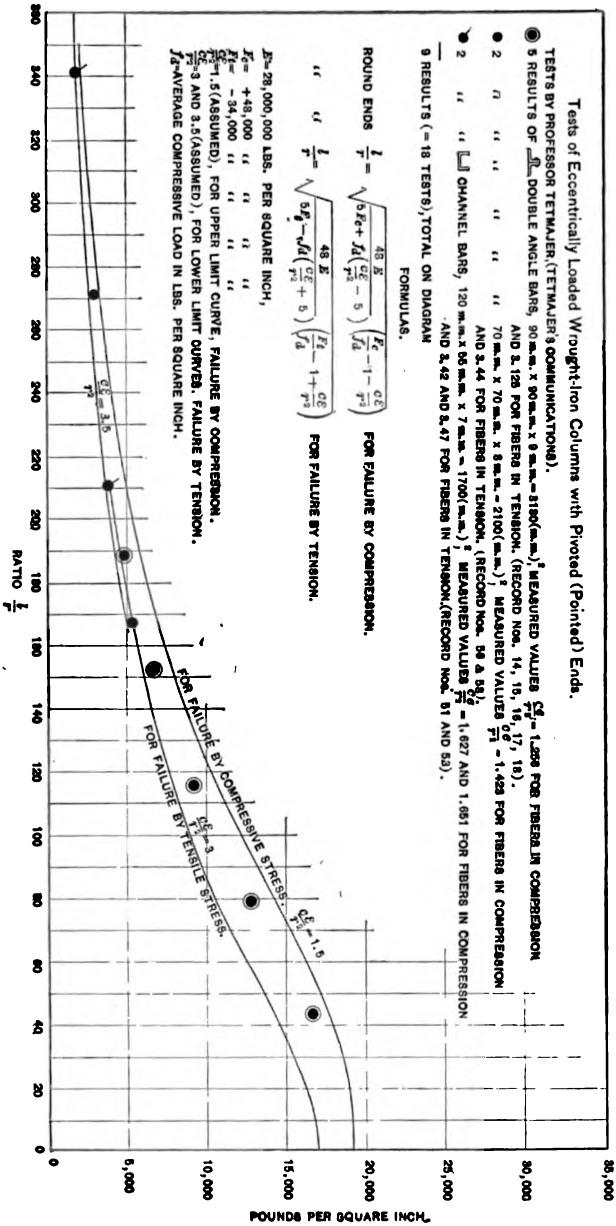

Tests of Eccentrically Loaded Wrought-Iron Columns with Pivoted (Pointed) Ends.
TESTS BY PROFESSOR TETMAJER.(TETMAJER'S COMMUNICATIONS).
5 RESULTS OF DOUBLE ANGLE BARS, 90 m.m. x 90 m.m. x 9 m.m. = 3180(m.m.)^2, MEASURED VALUES $\frac{ce}{r^2}$ = 1.268 FOR FIBERS IN COMPRESSION AND 3.125 FOR FIBERS IN TENSION. (RECORD Nos. 14, 15, 16, 17, 18).
2 " " " " " 70 m.m. x 70 m.m. x 8 m.m. = 2100(m.m.)^2 MEASURED VALUES $\frac{ce}{r^2}$ = 1.423 FOR FIBERS IN COMPRESSION AND 3.44 FOR FIBERS IN TENSION. (RECORD Nos. 59 & 58).
2 " " CHANNEL BARS, 120 m.m. x 56 m.m. x 7 m.m. = 1700(m.m.)^2, MEASURED VALUES $\frac{ce}{r^2}$ = 1.627 AND 1.651 FOR FIBERS IN COMPRESSION AND 3.42 AND 3.47 FOR FIBERS IN TENSION.(RECORD Nos. 51 AND 53).
9 RESULTS (= 18 TESTS),TOTAL ON DIAGRAM
FORMULAS.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48\,E}{5F_c + f_d\left(\frac{ce}{r^2} - 5\right)}}\left(\frac{F_c}{f_d} - 1 - \frac{ce}{r^2}\right)$ FOR FAILURE BY COMPRESSION.
" " $\frac{l}{r} = \sqrt{\frac{48\,E}{5F_t - f_d\left(\frac{ce}{r^2} + 5\right)}}\left(\frac{F_t}{f_d} - 1 + \frac{ce}{r^2}\right)$ FOR FAILURE BY TENSION.
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = +48,000 " " " "
F_t = −34,000 " " " "
$\frac{ce}{r^2}$ = 1.5(ASSUMED), FOR UPPER LIMIT CURVE, FAILURE BY COMPRESSION.
$\frac{ce}{r^2}$ = 3 AND 3.5(ASSUMED), FOR LOWER LIMIT CURVES. FAILURE BY TENSION.
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
$\frac{ce}{r^2}$ = 3.5
$\frac{ce}{r^2}$ = 3
$\frac{ce}{r^2}$ = 1.5
FOR FAILURE BY COMPRESSIVE STRESS.
FOR FAILURE BY TENSILE STRESS.
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000
POUNDS PER SQUARE INCH.

FIG. 94.

The record numbers of the results are noted on the diagram. Tetmajer's tests (non-axial), Nos. 27 to 34 inclusive, have not been plotted by the writer, as they were on T-bars, 100 mm. × 100 mm. × 10 mm., with the load imposed eccentrically in the line of the greatest radius of gyration, while the bars all failed in the direction of the least radius of gyration. It may be remarked that the amount of the intentional eccentricity in these tests (Nos. 27 to 34) was not sufficiently great in any case to ensure that failure would occur by flexure in the plane of the greatest radius of gyration, and the "accidental" equivalent eccentricity in the plane of the least radius of gyration was evidently the controlling factor.

The examples on the diagram have been selected for the sole reason that they were most nearly uniform in the character of section, method of loading, and amount of eccentricity.

Fourteen of the tests were on pairs of angle bars riveted together to form a T-section, and the remaining four were on channel bars.

In each case the eccentricity of loading was such that the tables of the T-sections or channels were subjected to the greatest compressive stress, and in consequence, the value of $\frac{c\,e}{r^2}$ was much less when referred to the table faces than when referred to the points of the legs of the T-sections or channels, rendering it necessary to use two different values of $\frac{c\,\varepsilon}{r^2}$, when plotting the curves by the writer's formula, one being for failure by compressive stress in the table faces, and the other for failure by tensile stress in the points of the legs.

The values of $\frac{c\,e}{r^2}$ deduced from the sections and the actual value of intentional eccentricity were as follows:

For results.	$\frac{c\,e}{r^2}$ for Compressive stress.	$\frac{c\,e}{r^2}$ for Tensile stress.
Nos. 14, 15, 16, 17 and 18..............	1.256	3.125
Nos. 56 and 58........................	1.428	3.440
Nos. 51 and 52........................	1.627 and 1.651	3.420 and 3.470

The curves plotted on the diagram have been calculated from values:

	For Compressive stress.	For Tensile stress.
$\frac{c\,\varepsilon}{r^2}$	1.5	3 and 3.5

and the characteristic agreement is sufficiently satisfactory when it is

remembered that we are dealing with tests of ultimate strength, and that each result plotted on the diagram represents the average of two tests.

The other non-axial tests made by Tetmajer had values $\frac{c\,e}{r^2}$ of too little uniformity to allow of them being used to illustrate the influence

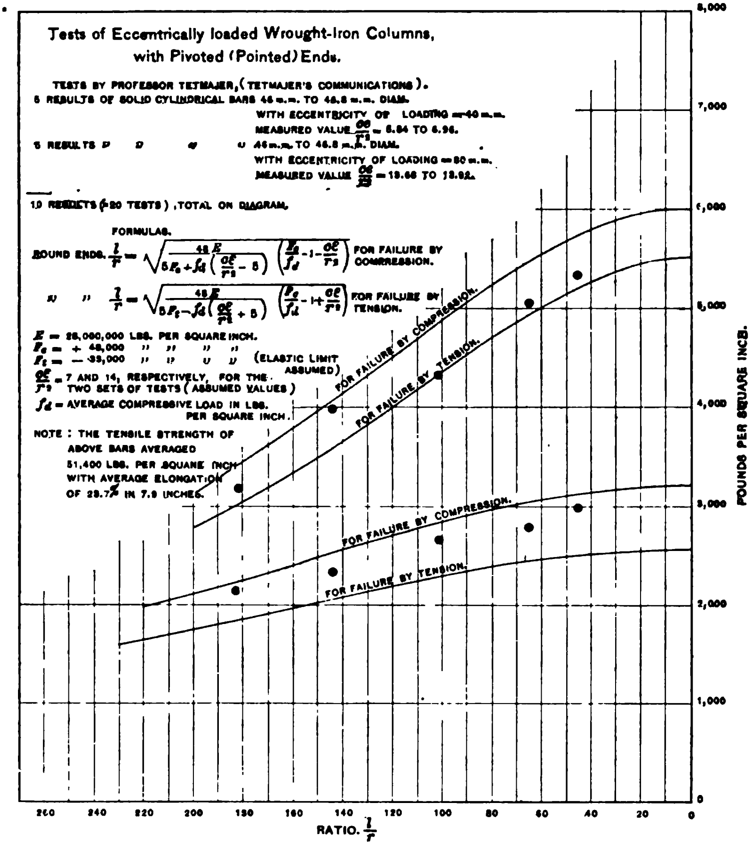


FIG. 25.

of eccentric loading combined with the influence of varying ratios of $\frac{l}{r}$. They do, however, most clearly show the serious loss of strength caused by eccentric loading.

Fig. 25.—Ten results representing twenty tests by Professor Tetmajer on pivot-ended wrought-iron columns under intentionally

eccentric loads. Each result plotted is the average of two tests.* These results are especially interesting, notwithstanding their small number, as the tests were on solid, round bars of one make of iron throughout, and the observed and intentional value of eccentricity of loading was very large, and thus greatly overshadowed accidental conditions.

Ten of the experiments were made with a value of $\frac{c\,e}{r^2} = 6.84$ to 6.96, and the other ten with a value of $\frac{c\,e}{r^2} = 13.68$ to 13.92. The ultimate strength of the iron under direct tension is given by Tetmajer as 51 400 lbs. per square inch, with an ultimate elongation of 23.7% in 200 mm. ($7\frac{7}{8}$ ins. nearly).

In this diagram the vertical scale of the load has been made much larger than in the other diagrams in order to emphasize the difference in the results, and to show their characteristic agreement with the writer's calculated curves.

In each of the two sets of experiments plotted on this diagram the upper curve indicates the loads causing a maximum compressive stress of 48 000 lbs. per square inch, and the lower curve indicates the loads causing a maximum tensile fiber stress of 36 000 lbs. per square inch.

The curves for the upper set of tests have been plotted for a value of $\frac{c\,\varepsilon}{r^2} = 7$, and those for the lower set for a value of 14.

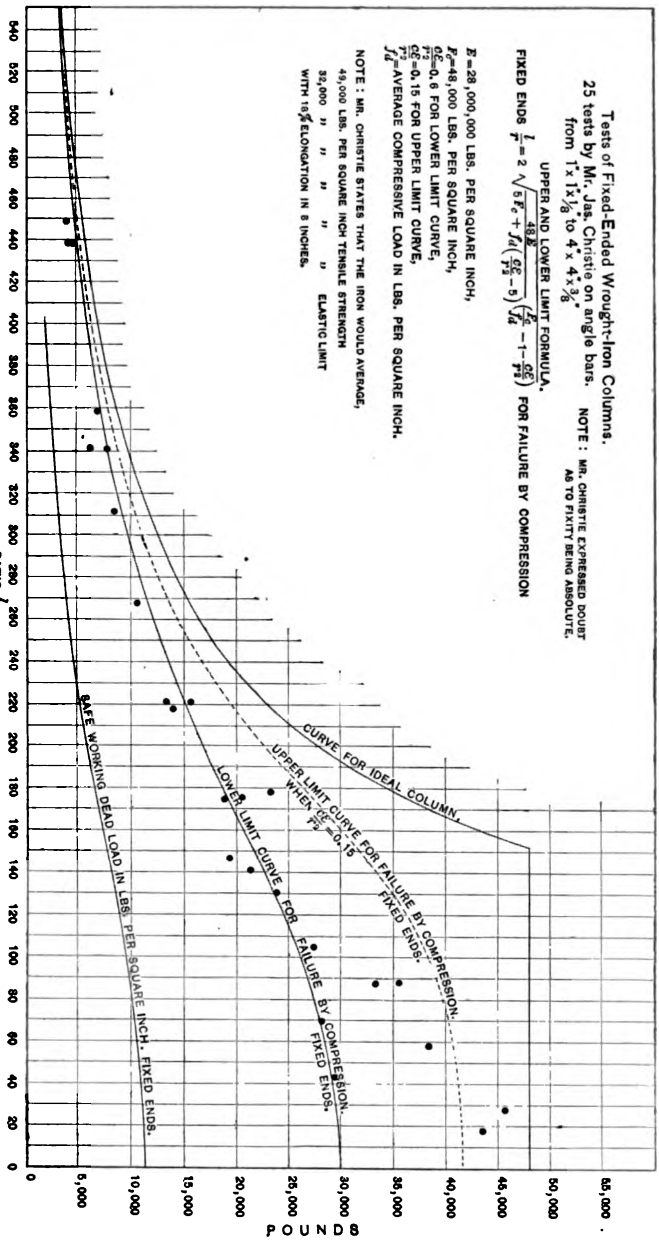
Wrought Iron.—Fixed Ends.

Fig. 26.—Twenty-five tests by Mr. Christie on fixed-ended columns of angle bars.† Reference has already been made to the difficulty of realizing fixity of ends in columns, and Mr. Christie remarks in his description of his experiments, that the lengths of the fixed-ended struts were measured between the clamps, whereas the point of absolute fixing probably occurred at some place within the clamps, and the values given for the ratio $\frac{l}{r}$ would then be too low.

If Mr. Christie's suggestion were adopted and a somewhat higher value assumed for the value of $\frac{l}{r}$, the already fairly satisfactory agreement between the writer's lower-limit curve and the lower results of the experiments would be still more pronounced.

* "Tetmajer's Communications."

† *Transactions*, Am. Soc. C. E., Vol. xiii.



Tests of Fixed-Ended Wrought-Iron Columns.
25 tests by Mr. Jas. Christie on angle bars.
from 1"x 1"x 1/8" to 4" x 4" x 3/8".
NOTE: MR. CHRISTIE EXPRESSED DOUBT AS TO FIXITY BEING ABSOLUTE.
UPPER AND LOWER LIMIT FORMULA.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48E}{5F_c + f_d\left(\frac{cc}{r^2} - 5\right)}}\left(\frac{F_c}{f_d} - 1 - \frac{cc}{r^2}\right)$ FOR FAILURE BY COMPRESSION
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 LBS. PER SQUARE INCH,
$\frac{cc}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{cc}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
NOTE: MR. CHRISTIE STATES THAT THE IRON WOULD AVERAGE,
49,000 LBS. PER SQUARE INCH TENSILE STRENGTH
32,000 " " " " ELASTIC LIMIT
WITH 18% ELONGATION IN 8 INCHES.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. WHEN $\frac{cc}{r^2}$ = 0.15 FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCH. FIXED ENDS.
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500 520 540
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000
POUNDS

FIG 26.

These experiments by Mr. Christie are the only series on fixed-ended columns of which the writer is aware. It is to be noted that Mr. Christie's Test No. 174, with ratio $\frac{l}{r} = 118$ (maximum load imposed being 24 050 lbs. per square inch), is not shown on the diagram, as failure did not take place.

Wrought Iron.—Flat-Ends.

Figs. 27, 28, 29, 30 and 31.—Tests of 240 flat-ended wrought-iron columns. The diagrams are self-explanatory, as far as possible. Attention is directed to the low results evidenced in these test of flat-ended columns when of considerable length, owing to their rotating on their ends. This mode of failure was found by Mr. Christie in his tests of flat-ended struts, as always occurring in the longest struts, and never in the shortest.

Fig. 32.—Results of seventy-nine tests of flat-ended wrought-iron columns of large size, of various sections, and by various experimenters. The writer has not been able to refer to the original records in every case, but the sources from which the information has been obtained are acknowledged on the diagram.

With regard to the Keystone columns tested by Mr. Bouscaren, only the four which were riveted through the projecting flanges, similarly to a Phœnix column, are recorded on the diagram.

Fig. 33.—Thirteen tests of wrought-iron flat-ended columns, by Professor Bauschinger.* These columns, which had flat ends, are not open to the objections raised against the use, as a basis of reference, of Bauschinger's pivot-ended columns.

Seven results, representing thirteen tests of flat-ended wrought-iron columns, by Professor Tetmajer†.

Twenty tests of flat-ended wrought-iron columns, by the late C. A. Marshall, M. Am. Soc. C. E.‡ Reference to Mr. Marshall's tests will be made subsequently.

Forty results = forty-six tests, total on diagram.

Fig. 34.—This diagram is compounded of Figs. 26 to 33, inclusive, and shows the results of 390 experiments (384 results), on wrought-iron columns, 25 of the tests being of fixed-ended columns, and 365 being of flat-ended columns. This diagram shows practi-

* Fünfzehntes Heft, *Mittheilung*, xviii.

† "Tetmajer's Communications."

‡ *Transactions*, Am. Soc. C. E., Vol. xvii.

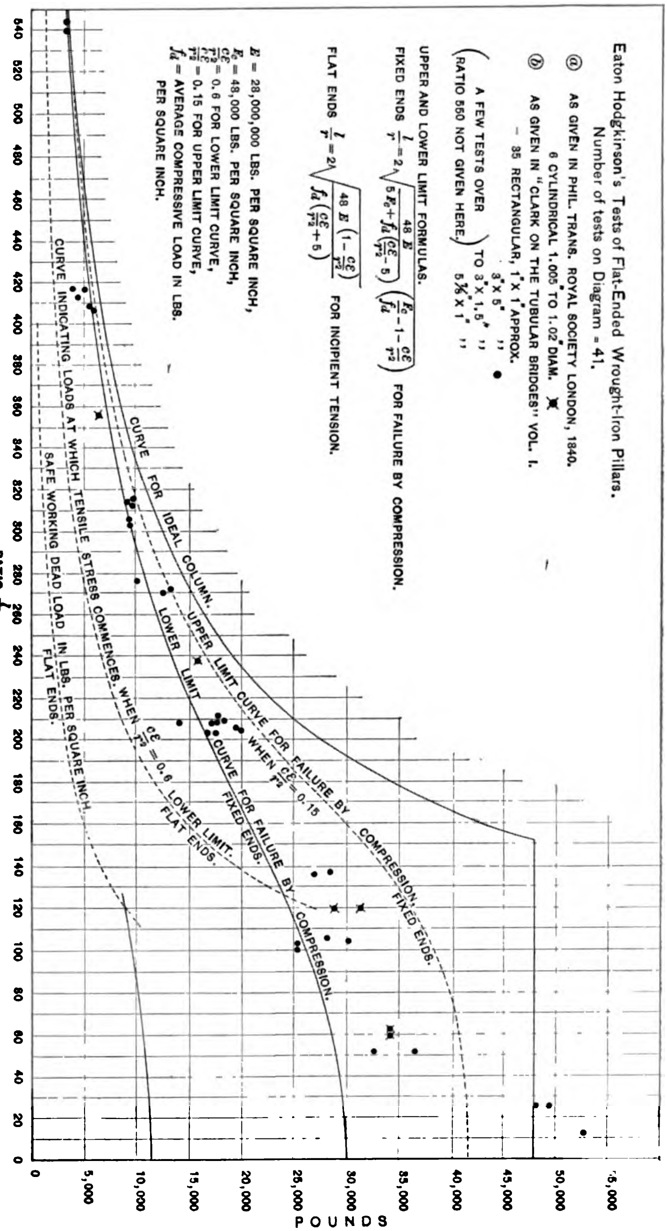

Eaton Hodgkinson's Tests of Flat-Ended Wrought-Iron Pillars.
Number of tests on Diagram = 41.
(a) AS GIVEN IN PHIL. TRANS. ROYAL SOCIETY LONDON, 1840.
6 CYLINDRICAL 1.005" TO 1.02" DIAM.
(b) AS GIVEN IN "CLARK ON THE TUBULAR BRIDGES" VOL. I.
– 35 RECTANGULAR, 1"x 1" APPROX.
TO 3"x 5" ,,
3"x 1.5" ,,
5⅝"x 1" ,,
A FEW TESTS OVER RATIO 550 NOT GIVEN HERE.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E}{5E_c + f_d\left(\frac{c\epsilon}{r^2}\cdot 5\right)}}\left(\frac{E_c}{f_d} - 1 - \frac{c\epsilon}{r^2}\right)$ FOR FAILURE BY COMPRESSION.
FLAT ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E\left(1 - \frac{c\epsilon}{r^2}\right)}{f_d\left(\frac{c\epsilon}{r^2} + 5\right)}}$ FOR INCIPIENT TENSION.
E = 28,000,000 LBS. PER SQUARE INCH,
E_c = 48,000 LBS. PER SQUARE INCH,
$\frac{c\epsilon}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{c\epsilon}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION, FIXED ENDS. WHEN $\frac{c\epsilon}{r^2}$ = 0.15
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
CURVE INDICATING LOADS AT WHICH TENSILE STRESS COMMENCES. WHEN $\frac{c\epsilon}{r^2}$ = 0.6 LOWER LIMIT. FLAT ENDS.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCH. FLAT ENDS.
540 520 500 480 460 440 420 400 380 360 340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000
POUNDS

FIG. 27.

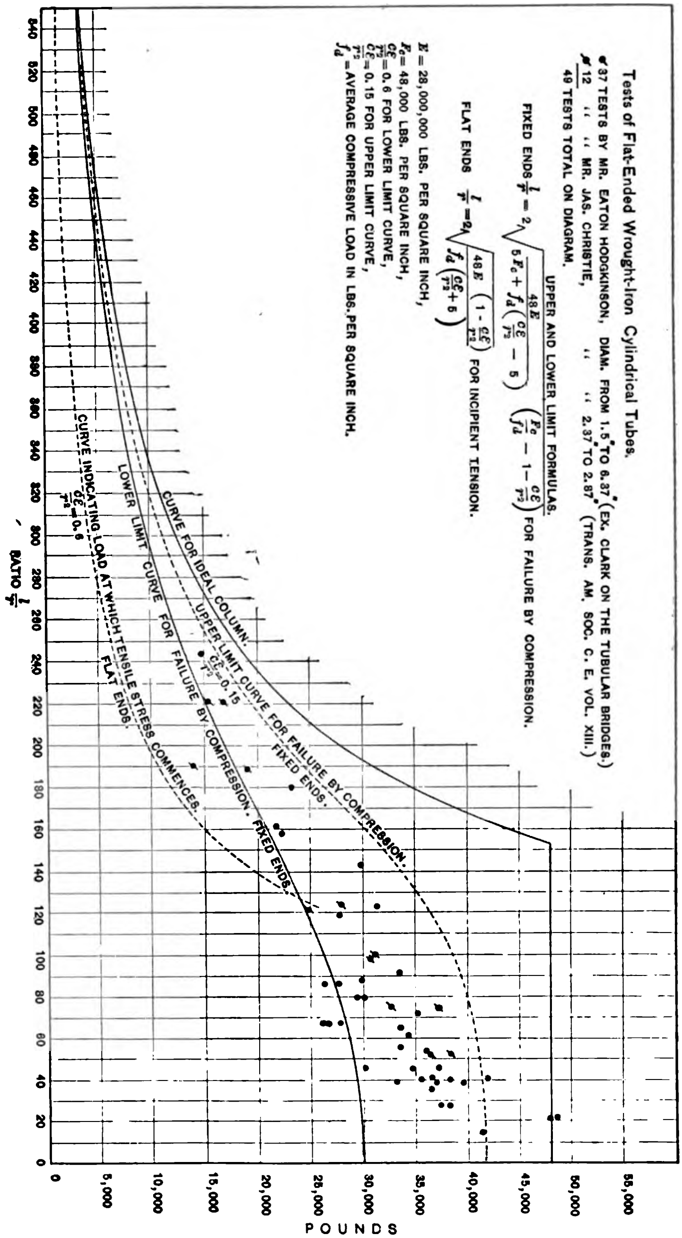

Tests of Flat-Ended Wrought-Iron Cylindrical Tubes.
● 37 TESTS BY MR. EATON HODGKINSON, DIAM. FROM 1.5" TO 6.37" (EX. CLARK ON THE TUBULAR BRIDGES.)
12 " " MR. JAS. CHRISTIE, " " 2.37" TO 2.87" (TRANS. AM. SOC. C. E. VOL. XIII.)
49 TESTS TOTAL ON DIAGRAM.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48E}{5F_c + f_d\left(\frac{c\epsilon}{r^2} - 5\right)}}\left(\frac{F_c}{f_d} - 1 - \frac{c\epsilon}{r^2}\right)$ FOR FAILURE BY COMPRESSION.
FLAT ENDS $\frac{l}{r} = 2\sqrt{\frac{48E\left(1 - \frac{c\epsilon}{r^2}\right)}{f_d\left(\frac{c\epsilon}{r^2} + 5\right)}}$ FOR INCIPIENT TENSION.
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 LBS. PER SQUARE INCH,
$\frac{c\epsilon}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{c\epsilon}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION.
$\frac{c\epsilon}{r^2}$ = 0.15
FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES.
$\frac{c\epsilon}{r^2}$ = 0.6
FLAT ENDS.
RATIO $\frac{l}{r}$
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500 520 540
POUNDS
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000

FIG. 38.

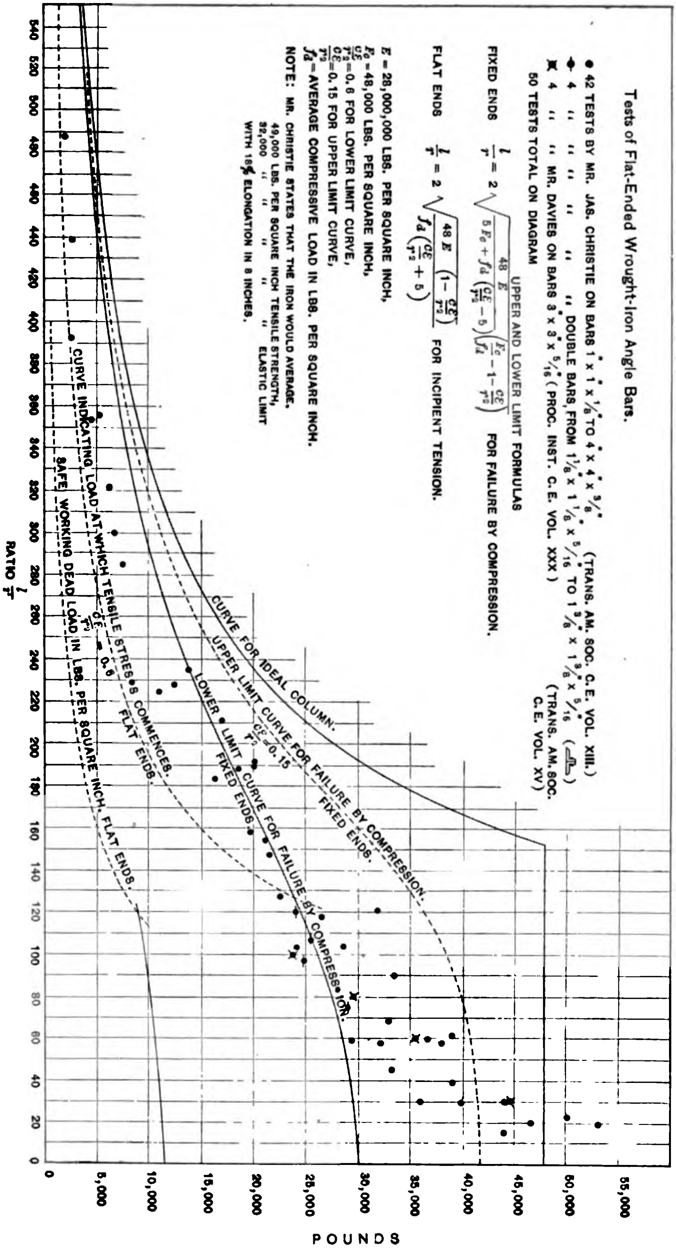

Tests of Flat-Ended Wrought-Iron Angle Bars.
● 42 TESTS BY MR. JAS. CHRISTIE ON BARS 1 x 1 x 1/8 TO 4 x 4 x 3/8 (TRANS. AM. SOC. C. E. VOL. XIII.)
◆ 4 " " " " " DOUBLE BARS, FROM 1 7/8 x 1 1/8 x 5/16 TO 1 3/8 x 1 3/8 x 5/16
✕ 4 " " MR. DAVIES ON BARS 3 x 3 x 5/16 (PROC. INST. C. E. VOL. XXX) (TRANS. AM. SOC. C. E. VOL. XV)
50 TESTS TOTAL ON DIAGRAM
UPPER AND LOWER LIMIT FORMULAS
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E}{5\,F_c + f_a\left(\frac{cE}{r^2} - 5\right)\left(\frac{F_c}{f_a} - 1 - \frac{cE}{r^2}\right)}}$ FOR FAILURE BY COMPRESSION.
FLAT ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E\left(1 - \frac{cE}{r^2}\right)}{f_a\left(\frac{cE}{r^2} + 5\right)}}$ FOR INCIPIENT TENSION.
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 LBS. PER SQUARE INCH,
$\frac{cE}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{cE}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_a = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
NOTE: MR. CHRISTIE STATES THAT THE IRON WOULD AVERAGE.
49,000 LBS. PER SQUARE INCH TENSILE STRENGTH,
32,000 " " " " ELASTIC LIMIT
WITH 18% ELONGATION IN 8 INCHES.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. $\frac{cE}{r^2}$ = 0.15 FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES. $\frac{cE}{r^2}$ = 0.6 FLAT ENDS.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCH. FLAT ENDS.
540 520 500 480 460 440 420 400 380 360 340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000
POUNDS

FIG. 30.

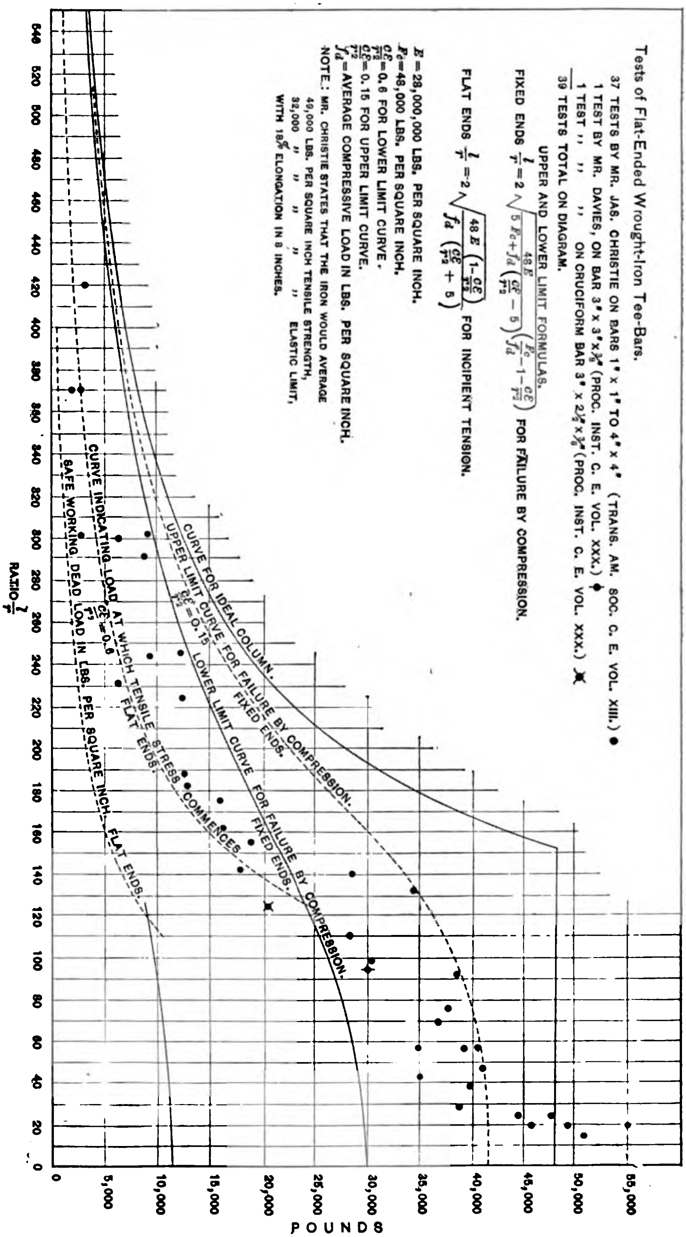

Tests of Flat-Ended Wrought-Iron Tee-Bars.
37 TESTS BY MR. JAS. CHRISTIE ON BARS 1" x 1" TO 4" x 4" (TRANS. AM. SOC. C. E. VOL. XIII.)
1 TEST BY MR. DAVIES, ON BAR 3" x 3" x 3/8" (PROC. INST. C. E. VOL. XXX.)
1 TEST " " " ON CRUCIFORM BAR 3" x 2 1/2" x 3/8" (PROC. INST. C. E. VOL. XXX.)
39 TESTS TOTAL ON DIAGRAM.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E}{5\,F_c + f_d\left(\frac{C\,c}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{C\,c}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
FLAT ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E\left(1 - \frac{C\,c}{r^2}\right)}{f_d\left(\frac{C\,c}{r^2} + 5\right)}}$ FOR INCIPIENT TENSION.
E = 28,000,000 LBS. PER SQUARE INCH.
F_c = 48,000 LBS. PER SQUARE INCH.
$\frac{C\,c}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE.
$\frac{C\,c}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE.
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
NOTE: MR. CHRISTIE STATES THAT THE IRON WOULD AVERAGE
49,000 LBS. PER SQUARE INCH TENSILE STRENGTH,
32,000 " " " " ELASTIC LIMIT,
WITH 18% ELONGATION IN 8 INCHES.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS. $\frac{C\,c}{r^2}$ = 0.15
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES. $\frac{C\,c}{r^2}$ = 0.6 FLAT ENDS.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCH. FLAT ENDS.
540 520 500 480 460 440 420 400 380 360 340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000
POUNDS

FIG. 80.

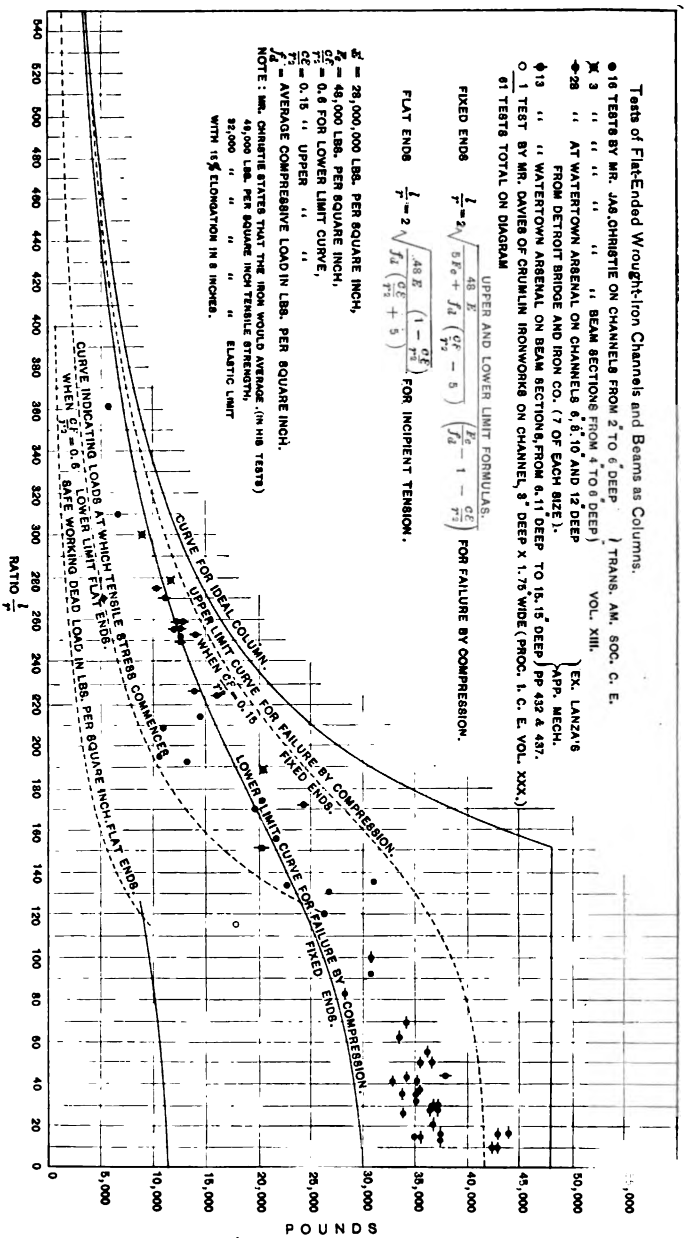

Tests of Flat-Ended Wrought-Iron Channels and Beams as Columns.
● 16 TESTS BY MR. JAS. CHRISTIE ON CHANNELS FROM 2" TO 6" DEEP } TRANS. AM. SOC. C. E.
3 " " " " BEAM SECTIONS FROM 4" TO 6" DEEP } VOL. XIII.
28 " AT WATERTOWN ARSENAL ON CHANNELS 6, 8, 10 AND 12" DEEP FROM DETROIT BRIDGE AND IRON CO. (7 OF EACH SIZE).
13 " " WATERTOWN ARSENAL ON BEAM SECTIONS, FROM 6.11" DEEP TO 15.15" DEEP } EX. LANZA'S APP. MECH. PP 432 & 437.
○ 1 TEST BY MR. DAVIES OF CRUMLIN IRONWORKS ON CHANNEL, 8" DEEP X 1.75" WIDE (PROC. I. C. E. VOL. XXX.)
61 TESTS TOTAL ON DIAGRAM
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48\ E}{5F_e + f_d\left(\frac{cE}{r^2} - 5\right)}\left(\frac{F_e}{f_d} - 1 - \frac{cE}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
FLAT ENDS $\frac{l}{r} = 2\sqrt{\frac{.48\ E\left(1 - \frac{cE}{r^2}\right)}{f_d\left(\frac{cE}{r^2} + 5\right)}}$ FOR INCIPIENT TENSION.
E = 28,000,000 LBS. PER SQUARE INCH.
F_e = 48,000 LBS. PER SQUARE INCH.
$\frac{cE}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE.
$\frac{cE}{r^2}$ = 0.15 " UPPER " "
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
NOTE: MR. CHRISTIE STATES THAT THE IRON WOULD AVERAGE (IN HIS TESTS)
49,000 LBS. PER SQUARE INCH TENSILE STRENGTH.
32,000 " " " " ELASTIC LIMIT
WITH 15% ELONGATION IN 8 INCHES.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE WHEN $\frac{cE}{r^2}$ = 0.15
CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
CURVE INDICATING LOADS AT WHICH TENSILE STRESS COMMENCES WHEN $\frac{cE}{r^2}$ = 0.6 LOWER LIMIT FLAT ENDS.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCH. FLAT ENDS
540 520 500 480 460 440 420 400 380 360 340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000
POUNDS

FIG. 81.

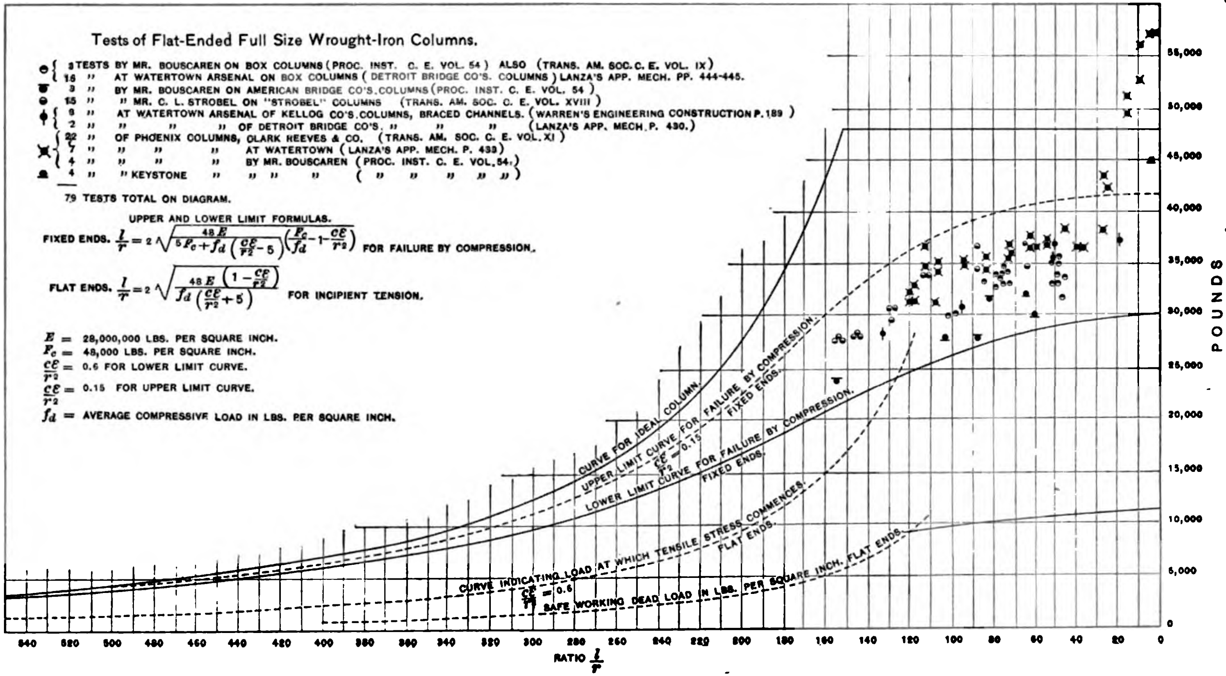
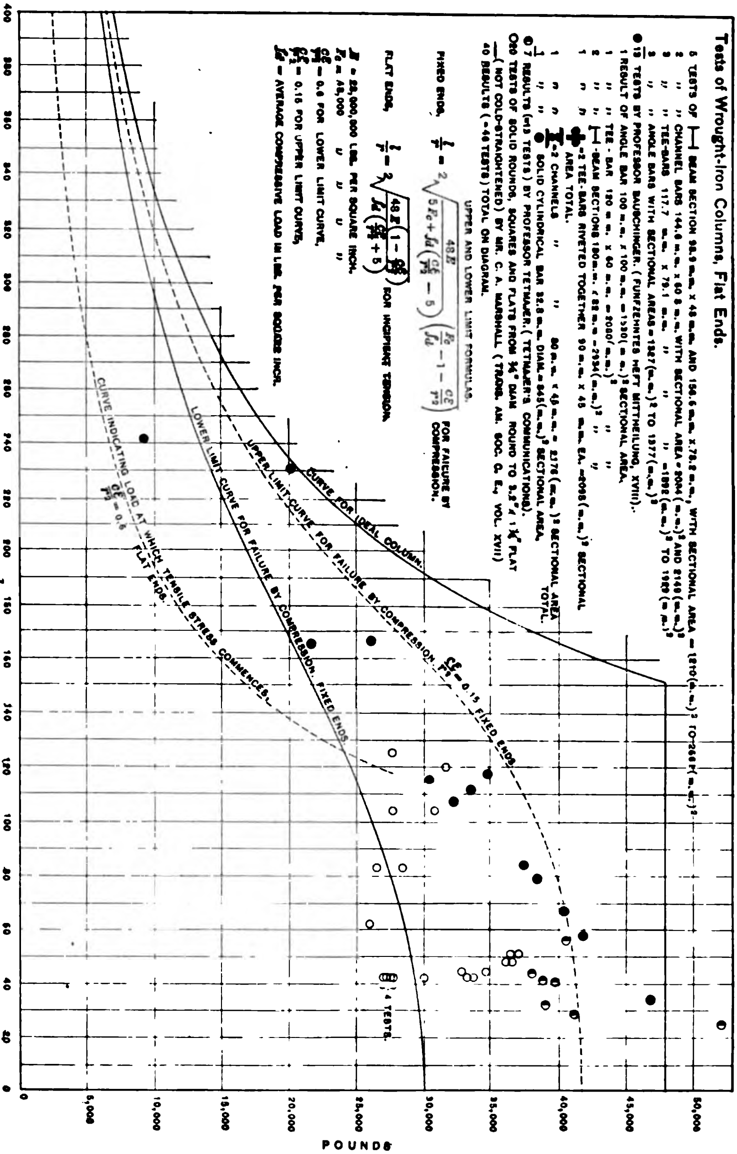

Tests of Flat-Ended Full Size Wrought-Iron Columns.
3 TESTS BY MR. BOUSCAREN ON BOX COLUMNS (PROC. INST. C. E. VOL. 54) ALSO (TRANS. AM. SOC. C. E. VOL. IX)
16 " AT WATERTOWN ARSENAL ON BOX COLUMNS (DETROIT BRIDGE CO'S. COLUMNS) LANZA'S APP. MECH. PP. 444-445.
3 " BY MR. BOUSCAREN ON AMERICAN BRIDGE CO'S. COLUMNS (PROC. INST. C. E. VOL. 54)
15 " " MR. C. L. STROBEL ON "STROBEL" COLUMNS (TRANS. AM. SOC. C. E. VOL. XVIII)
9 " AT WATERTOWN ARSENAL OF KELLOG CO'S. COLUMNS, BRACED CHANNELS. (WARREN'S ENGINEERING CONSTRUCTION P. 189)
2 " " " " OF DETROIT BRIDGE CO'S. " " " (LANZA'S APP. MECH. P. 430.)
22 " OF PHOENIX COLUMNS, CLARK REEVES & CO. (TRANS. AM. SOC. C. E. VOL. XI)
7 " " " " AT WATERTOWN (LANZA'S APP. MECH. P. 433)
4 " " " " BY MR. BOUSCAREN (PROC. INST. C. E. VOL. 54.)
4 " " KEYSTONE " " " " (" " " " ")
79 TESTS TOTAL ON DIAGRAM.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS. $\frac{l}{r} = 2\sqrt{\frac{48E}{5F_c + f_d\left(\frac{cE}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{cE}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
FLAT ENDS. $\frac{l}{r} = 2\sqrt{\frac{48E\left(1 - \frac{cE}{r^2}\right)}{f_d\left(\frac{cE}{r^2} + 5\right)}}$ FOR INCIPIENT TENSION.
E = 28,000,000 LBS. PER SQUARE INCH.
F_c = 48,000 LBS. PER SQUARE INCH.
$\frac{cE}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE.
$\frac{cE}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE.
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. $\frac{cE}{r^2} = 0.15$ FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES. $\frac{cE}{r^2} = 0.6$ FLAT ENDS.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCH. FLAT ENDS.
340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000
POUNDS

FIG. 38.



RATIO $\frac{l}{r}$

FIG. 38.

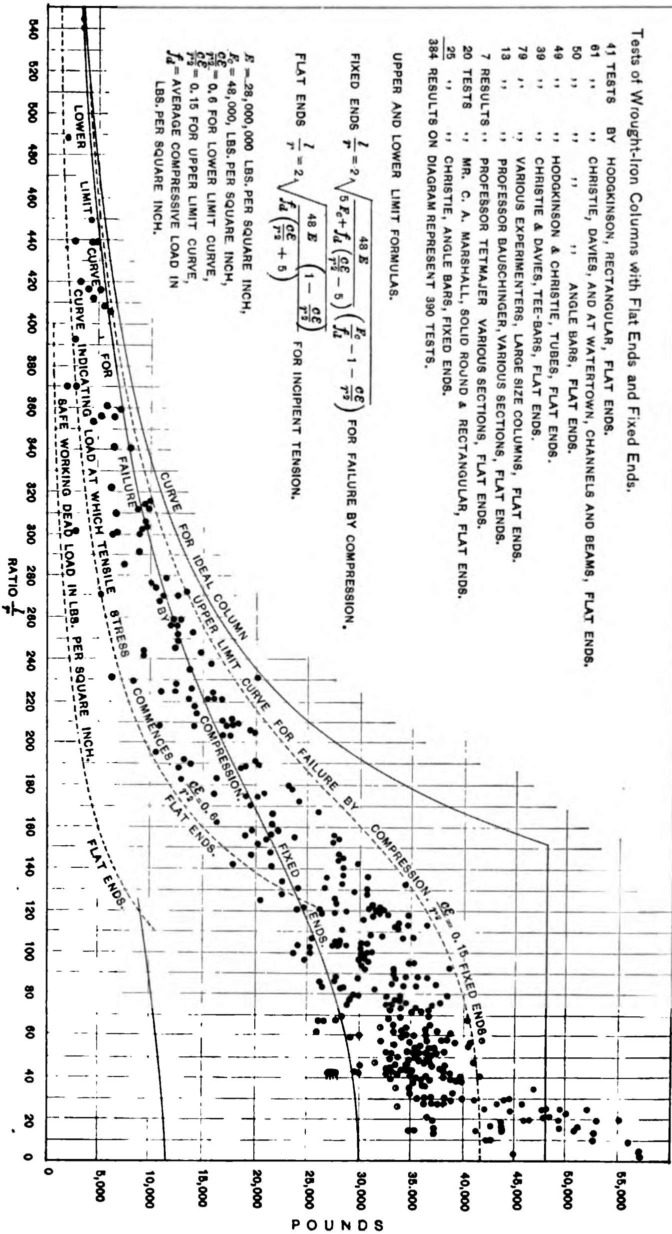

Tests of Wrought-Iron Columns with Flat Ends and Fixed Ends.
41 TESTS BY HODGKINSON, RECTANGULAR, FLAT ENDS.
61 " " CHRISTIE, DAVIES, AND AT WATERTOWN, CHANNELS AND BEAMS, FLAT ENDS.
50 " " " " ANGLE BARS, FLAT ENDS.
49 " " HODGKINSON & CHRISTIE, TUBES, FLAT ENDS.
39 " " CHRISTIE & DAVIES, TEE-BARS, FLAT ENDS.
79 " " VARIOUS EXPERIMENTERS, LARGE SIZE COLUMNS, FLAT ENDS.
13 " " PROFESSOR BAUSCHINGER, VARIOUS SECTIONS, FLAT ENDS.
7 RESULTS " PROFESSOR TETMAJER VARIOUS SECTIONS, FLAT ENDS.
20 TESTS " MR. C. A. MARSHALL, SOLID ROUND & RECTANGULAR, FLAT ENDS.
25 " " CHRISTIE, ANGLE BARS, FIXED ENDS.
384 RESULTS ON DIAGRAM REPRESENT 390 TESTS.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E}{5\,F_c + f_d\left(\frac{c\,\epsilon}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{c\,\epsilon}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
FLAT ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E}{f_d\left(\frac{c\,\epsilon}{r^2} + 5\right)}\left(1 - \frac{c\,\epsilon}{r^2}\right)}$ FOR INCIPIENT TENSION.
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000, LBS. PER SQUARE INCH,
$\frac{c\epsilon}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{c\epsilon}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. $\frac{c\epsilon}{r^2}$ = 0.15. FIXED ENDS.
CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE STRESS COMMENCES. $\frac{c\epsilon}{r^2}$ = 0.6. FLAT ENDS.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCH. FLAT ENDS.
540 520 500 480 460 440 420 400 380 360 340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000
POUNDS

FIG. 34.

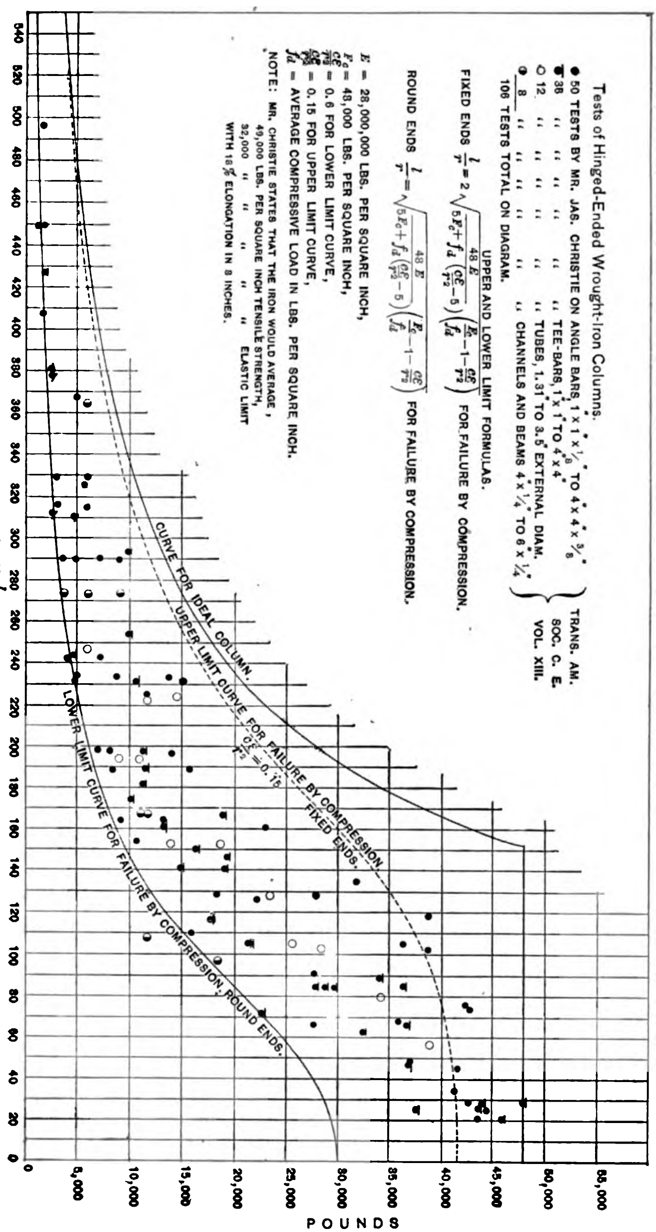

Tests of Hinged-Ended Wrought-Iron Columns.
● 50 TESTS BY MR. JAS. CHRISTIE ON ANGLE BARS, 1" x 1" x 1/8" TO 4" x 4" x 3/8"
38 " " " " " TEE-BARS, 1" x 1" TO 4" x 4"
○ 12 " " " " " TUBES, 1.31" TO 3.5" EXTERNAL DIAM.
8 " " " " " CHANNELS AND BEAMS 4" x 1/4" TO 6" x 1/4"
106 TESTS TOTAL ON DIAGRAM.
TRANS. AM. SOC. C. E. VOL. XIII.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E}{5F_c + f_d\left(\frac{cC}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{cC}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48\,E}{5F_c + f_d\left(\frac{cC}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{cC}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 LBS. PER SQUARE INCH,
$\frac{cC}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{cC}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
NOTE: MR. CHRISTIE STATES THAT THE IRON WOULD AVERAGE,
49,000 LBS. PER SQUARE INCH TENSILE STRENGTH,
32,000 " " " " ELASTIC LIMIT
WITH 18% ELONGATION IN 8 INCHES.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION, FIXED ENDS. $\frac{cC}{r^2} = 0.15$
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION, ROUND ENDS.
RATIO $\frac{l}{r}$
POUNDS
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460
480
500
520
540
0
5,000
10,000
15,000
20,000
25,000
30,000
35,000
40,000
45,000
50,000
55,000

FIG. 35.

cally all the available experimental data on the strength of wrought-iron columns with flat ends.

The weakness of flat-ended columns in the higher ratios of $\frac{l}{r}$ is most clearly evident in this diagram. The high strength shown by a number of columns of ratio $\frac{l}{r} = 30$, and under, is also a striking feature, and it would appear that columns longer than 30 radii of gyration ($7\frac{1}{2}$ diameters in cylindrical solid bars) cannot be expected to develop higher strength due to plastic yielding under compression, and that column action begins to come into play more definitely at about ratios $\frac{l}{r} = 30$ to 40.

It is evident that in very short columns the useful ultimate strength must be measured by the elastic limit under compression, and this may not have been noted carefully in some of the experiments, but beyond the ratio 30, the influence of column length appears to become very strongly marked.

Wrought Iron.—Hinged or Pin Ends.

Fig. 35.—One hundred and six tests of hinged-ended angles, tees, tubes, channels and beams, by Mr. Christie.* In Mr. Christie's account of his tests, he remarks: "The hinged-ended tests varied all the way from the value of round-ended up to flat-ended."

The writer would extend this to, "the hinged-ended tests varied all the way from the lower values for round-ended up to the higher values for flat or fixed-ended."

The truth of this is apparent on referring to the diagram on which the writer has plotted the lower limit curve for round-ended columns with $\frac{c\,\varepsilon}{r^2} = 0.6$, as on Figs. 20, 21 and 22, and the upper-limit dotted curve for fixed-ended columns with $\frac{c\,\varepsilon}{r^2} = 0.15$, as on Figs. 26 to 33.

The high results for ratios under 30 are again evident, and are similar to the results for flat-ended columns.

The tests on hinged-ended angles, given in Table No. 3 of Mr. Christie's paper, are all plotted on the diagram, excepting those which were not carried to the point of failure. In addition to these, the writer has also plotted five of Mr. Christie's extra tests on some of these angle bars, as given on pages 113 and 114 of Mr. Christie's paper.

**Transactions*, Am. Soc. C. E., Vol. xiii.

These are as follows:

1. First test, with 2-in. balls and sockets, apparently central, on the same bar as in Experiment No. 80, ratio $\frac{l}{r} = 164$, for which the result of the second test after moving the ends 0.06 in. from the original position was given in Mr. Christie's Table No. 3. The first test, apparently central, gave an ultimate strength of 19 400 lbs. (9 200 lbs. per square inch), and the second test, after moving the specimen 0.06 in., gave an ultimate strength of 27 850 lbs. (13 199 lbs. per square inch), Both of these results are on the diagram.

2 and 3. The second and third tests on the same bar as in Experiment No. 117, ratio $\frac{l}{r} = 290$. The first test (No. 117) on this bar was with 2-in. balls and sockets (7 020 lbs. per square inch), the second with 1-in. balls and sockets (3 525 lbs. per square inch), and the third with 2-in. pins. (4 790 lbs. per square inch).

4. The second test on the same bar as in Experiment No. 118, ratio $\frac{l}{r} = 233$. In the case of this bar, the first test result, with ends apparently central, is given in Mr. Christie's Table No. 3, the strength being 16 700 lbs. (8 650 lbs. per square inch), and in the second test the ends were moved 0.06 in., with the result that the strength was 26 450 lbs. (13 700 lbs. per square inch).

5. The first test on the same bar as in Experiment No. 119, ratio $\frac{l}{r} = 189$, giving a strength of 20 150 lbs. (8 340 lbs. per square inch). In the case of this bar, the second test, after moving the bar 0.07 in., with resulting ultimate strength of 38 175 lbs. (15 770 lbs. per square inch), is that given in Mr. Christie's Table No. 3.

It is not clear why Mr. Christie should have selected the second tests in the case of Experiments Nos. 80 and 119, for insertion in his Table No. 3, which otherwise referred to struts under apparently central loads.

Mr. Christie's extra tests, of considerable number, on angles, tees, tubes, and H-beams, in addition to the few above mentioned, made it most apparent that the physical axis did not always coincide with the geometric axis, and they also showed very markedly how very sensitive the columns were to apparently insignificant amounts of adjustment, and to variations in sizes of pin or socket.

Fig. 36.—One hundred and twenty-six tests of pin-ended wrought-iron columns of large size by various experimenters. The writer has not had the advantage of referring to the original published records of most of the tests, and he has therefore been compelled to take the records at second hand from the sources acknowledged on the diagram.

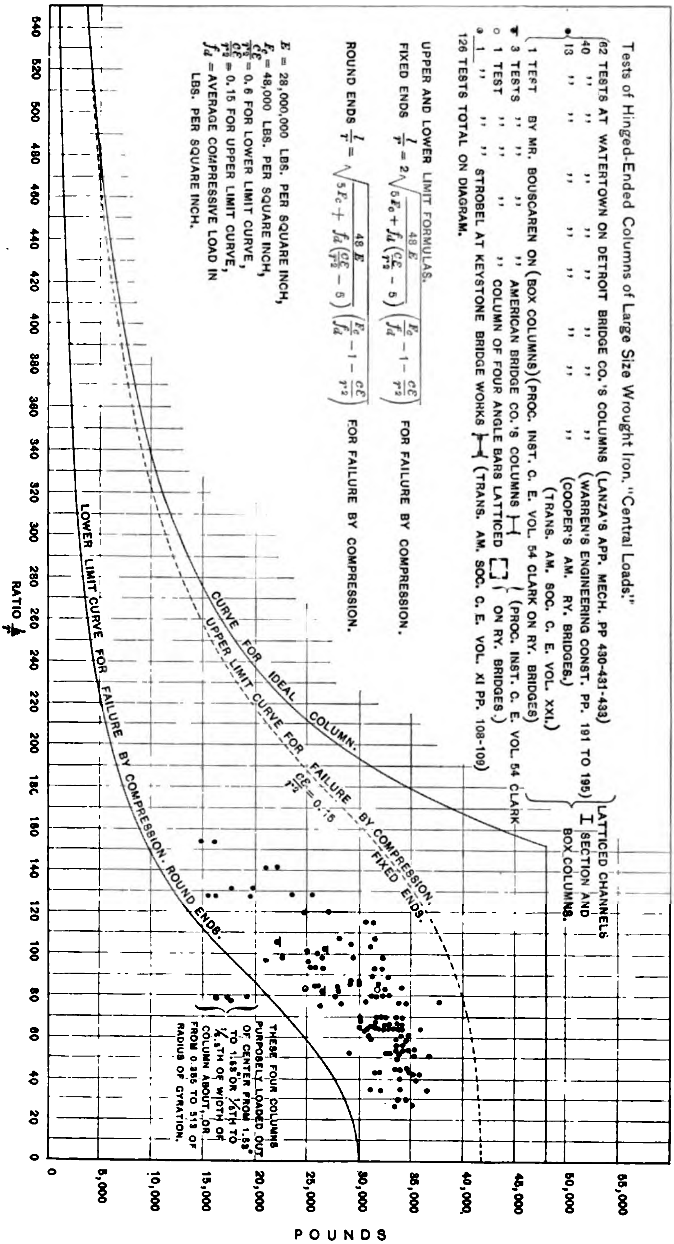
With regard to the Detroit Bridge Company's columns, the results have been taken from the paper by Theodore Cooper, M. Am. Soc. C. E.,* from Professor Lanza's "Applied Mechanics," and Professor W. H. Warren's "Engineering Construction."

The results, in many cases, if plotted to the values of $\frac{l}{r}$ given by Mr. Cooper, would differ considerably from the results as plotted on the diagram. This results from the writer's having found that the values of the ratio $\frac{l}{r}$, given by Mr. Cooper, do not agree with the dimensions of the columns as given by Professor Lanza and Professor Warren. The writer, accordingly, recalculated the values of $\frac{l}{r}$, using the lengths as given by Professors Lanza and Warren, and, calculating the radius of gyration from the detailed sections given on Plate XXVII of Mr. Cooper's paper, at the same time comparing carefully the sections as given by Mr. Cooper with the less fully detailed sections given by Professors Lanza and Warren.

It would appear that Mr. Cooper has taken the least radius of gyration in every case, but these columns were pin-ended, in which the radius of gyration at right angles to the pin was frequently greater than that parallel to the pin, and it is this greater radius of gyration which should be used, as the columns were practically flat-ended as regards failure in the direction of the least radius of gyration. The effect of this is that Mr. Cooper's values of $\frac{l}{r}$ are in many cases too great, and, consequently, too great strength has been credited to these high values of $\frac{l}{r}$.

It is worthy of note that a few of these columns did fail by deflecting in a direction parallel to the pins, and acting as flat-ended columns, and not as pin-ended. This draws attention to the necessity for care in proportioning pin-ended columns in which the radius of

* "American Railway Bridges," *Transactions*, Am. Soc. C. E., Vol. xxi.



Tests of Hinged-Ended Columns of Large Size Wrought Iron. "Central Loads."
62 TESTS AT WATERTOWN ON DETROIT BRIDGE CO.'S COLUMNS (LANZA'S APP. MECH. PP 430-431-433)
40 " " " " " " " (WARREN'S ENGINEERING CONST. PP. 191 TO 195)
13 " " " " " " " (COOPER'S AM. RY. BRIDGES.) (TRANS. AM. SOC. C. E. VOL. XXI.)
LATTICED CHANNELS I SECTION AND BOX COLUMNS.
1 TEST BY MR. BOUSCAREN ON (BOX COLUMNS)(PROC. INST. C. E. VOL. 54 CLARK ON RY. BRIDGES)
3 TESTS " " " AMERICAN BRIDGE CO.'S COLUMNS (PROC. INST. C. E. VOL. 54 CLARK
1 TEST " " " COLUMN OF FOUR ANGLE BARS LATTICED ON RY. BRIDGES.)
1 " " " STROBEL AT KEYSTONE BRIDGE WORKS (TRANS. AM. SOC. C. E. VOL. XI PP. 108-109)
126 TESTS TOTAL ON DIAGRAM.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48\,E}{5E_c + f_d\left(\frac{cE}{r^2} - 5\right)}\left(\frac{E_c}{f_d} - 1 - \frac{cE}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48\,E}{5E_c + f_d\left(\frac{cE}{r^2} - 5\right)}\left(\frac{E_c}{f_d} - 1 - \frac{cE}{r^2}\right)}$ FOR FAILURE BY COMPRESSION.
E = 28,000,000 LBS. PER SQUARE INCH,
E_c = 48,000 LBS. PER SQUARE INCH,
$\frac{cE}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{cE}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
$\frac{cE}{r^2}$ = 0.15
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. ROUND ENDS.
THESE FOUR COLUMNS PURPOSELY LOADED OUT OF CENTER FROM 1.5" TO 1.63" OR 1/5TH TO 1/6TH OF WIDTH OF COLUMN ABOUT, OR FROM 0.385 TO 513 OF RADIUS OF GYRATION.
540 520 500 480 460 440 420 400 380 360 340 320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 50,000 55,000
POUNDS

FIG. 36.

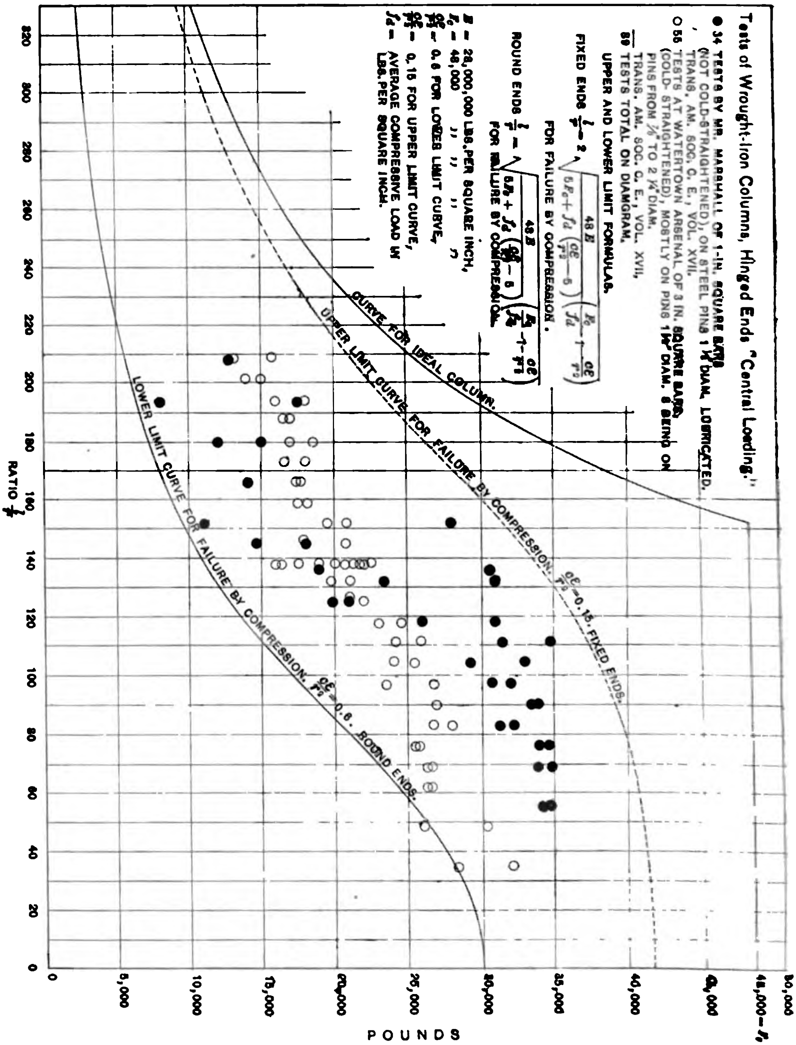

Tests of Wrought-Iron Columns, Hinged Ends "Central Loading."
● 34 TESTS BY MR. MARSHALL OF 1-IN. SQUARE BARS (NOT COLD-STRAIGHTENED), ON STEEL PINS 1 ½" DIAM. LUBRICATED. TRANS. AM. SOC. C. E., VOL. XVII.
○ 55 TESTS AT WATERTOWN ARSENAL OF 3 IN. SQUARE BARS (COLD-STRAIGHTENED), MOSTLY ON PINS 1 ½" DIAM. 8 BEING ON PINS FROM ⅞" TO 2 ¼" DIAM. TRANS. AM. SOC. C. E., VOL. XVII.
89 TESTS TOTAL ON DIAGRAM.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48E}{5F_c + f_d\left(\frac{c\mathcal{E}}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{c\mathcal{E}}{r^2}\right)}$
FOR FAILURE BY COMPRESSION.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48E}{5F_c + f_d\left(\frac{c\mathcal{E}}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{c\mathcal{E}}{r^2}\right)}$
FOR FAILURE BY COMPRESSION.
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 " " "
$\frac{c\mathcal{E}}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{c\mathcal{E}}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. $\frac{c\mathcal{E}}{r^2}$ = 0.15. FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. $\frac{c\mathcal{E}}{r^2}$ = 0.6. ROUND ENDS.
50,000
48,000 — F_c
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
POUNDS
320 300 280 260 240 220 200 180 160 140 120 100 80 60 40 20 0
RATIO $\frac{l}{r}$

Fig. 37.

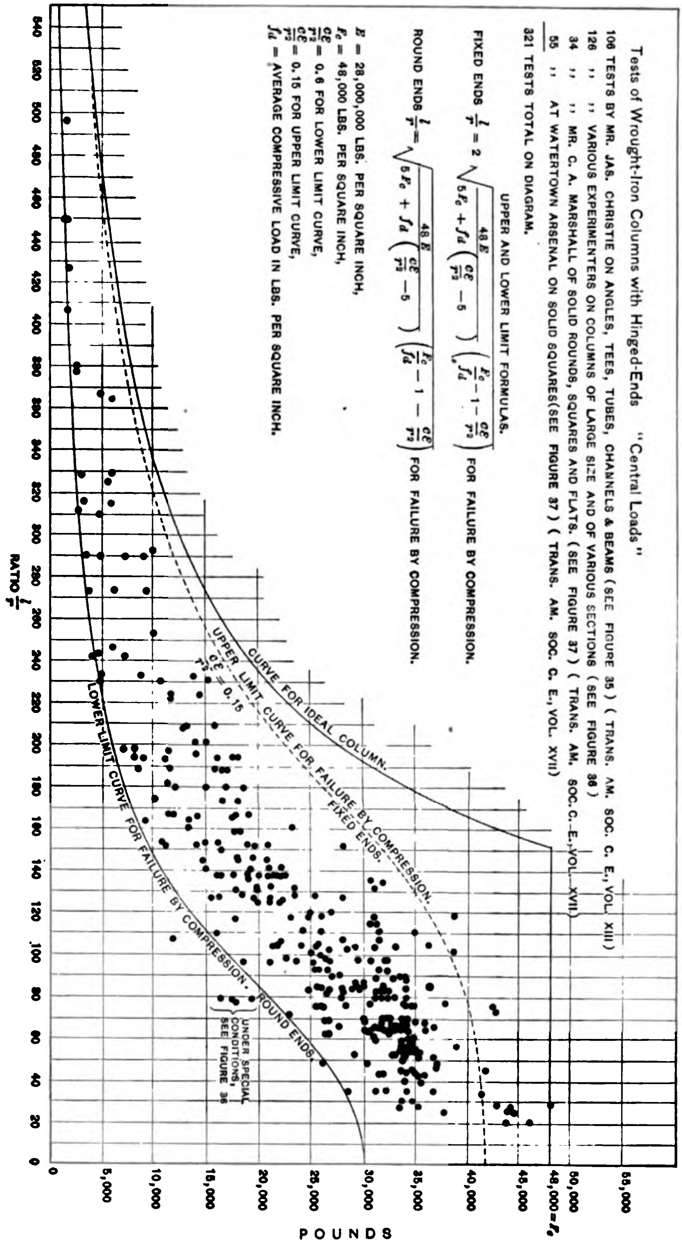

Tests of Wrought-Iron Columns with Hinged-Ends "Central Loads"
106 TESTS BY MR. JAS. CHRISTIE ON ANGLES, TEES, TUBES, CHANNELS & BEAMS (SEE FIGURE 35) (TRANS. AM. SOC. C. E., VOL. XIII)
126 " " VARIOUS EXPERIMENTERS ON COLUMNS OF LARGE SIZE AND OF VARIOUS SECTIONS (SEE FIGURE 36)
34 " " MR. C. A. MARSHALL OF SOLID ROUNDS, SQUARES AND FLATS. (SEE FIGURE 37) (TRANS. AM. SOC. C. E., VOL. XVII)
55 " AT WATERTOWN ARSENAL ON SOLID SQUARES (SEE FIGURE 37) (TRANS. AM. SOC. C. E., VOL. XVII)
321 TESTS TOTAL ON DIAGRAM.
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS $\frac{l}{r} = 2\sqrt{\frac{48E}{5F_c + f_d\left(\frac{cE}{r^2} - 5\right)}}\left(\frac{F_c}{f_d} - 1 - \frac{cE}{r^2}\right)$ FOR FAILURE BY COMPRESSION.
ROUND ENDS $\frac{l}{r} = \sqrt{\frac{48E}{5F_c + f_d\left(\frac{cE}{r^2} - 5\right)}}\left(\frac{F_c}{f_d} - 1 - \frac{cE}{r^2}\right)$ FOR FAILURE BY COMPRESSION.
E = 28,000,000 LBS. PER SQUARE INCH,
F_c = 48,000 LBS. PER SQUARE INCH,
$\frac{cE}{r^2}$ = 0.6 FOR LOWER LIMIT CURVE,
$\frac{cE}{r^2}$ = 0.15 FOR UPPER LIMIT CURVE,
f_d = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. FIXED ENDS.
$\frac{cE}{r^2}$ = 0.15
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION. ROUND ENDS.
UNDER SPECIAL CONDITIONS, SEE FIGURE 36
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460 480 500 520 540
0 5,000 10,000 15,000 20,000 25,000 30,000 35,000 40,000 45,000 48,000 = F_c 50,000 55,000
RATIO $\frac{l}{r}$
POUNDS

FIG. 38.

gyration, parallel to the axis of the pins, is of less value than the radius perpendicular to the pins.

If too great a difference exists. the column may be weaker as a flat-ended column deflecting parallel to the pins than as a pin-ended column deflecting at right angles to the pins.

Four of the Detroit Bridge Company's columns were purposely loaded out of axis, and these are plotted on the diagram with a note to that effect. Two of the columns were formed of a pair of 10-in.

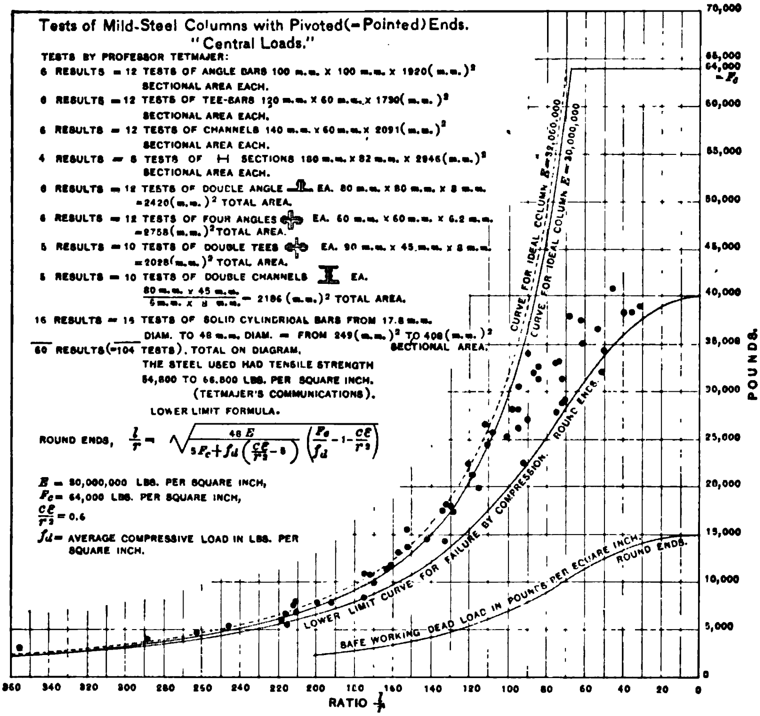


FIG. 39.

channels connected on one side by a plate, and on the other by latticing. The other two were of the same type, but with 8-in. channels.

The low strengths are worthy of careful attention, as the results were evidently not entirely due to these four columns being of unsymmetrical section, but to the pins not being placed at the center of area, since the highest result shown on the whole diagram was of the same type and proportions as these four columns, but with pins placed at the center of area.

Fig. 37.—This diagram represents:

Thirty-four tests of pin-ended wrought-iron columns by Mr. C. A. Marshall;* and fifty-five tests of pin-ended wrought-iron columns at Watertown,† a total of eighty-nine tests. A reference to these will be made when dealing with Mr. Marshall's tests on pin-ended steel columns.

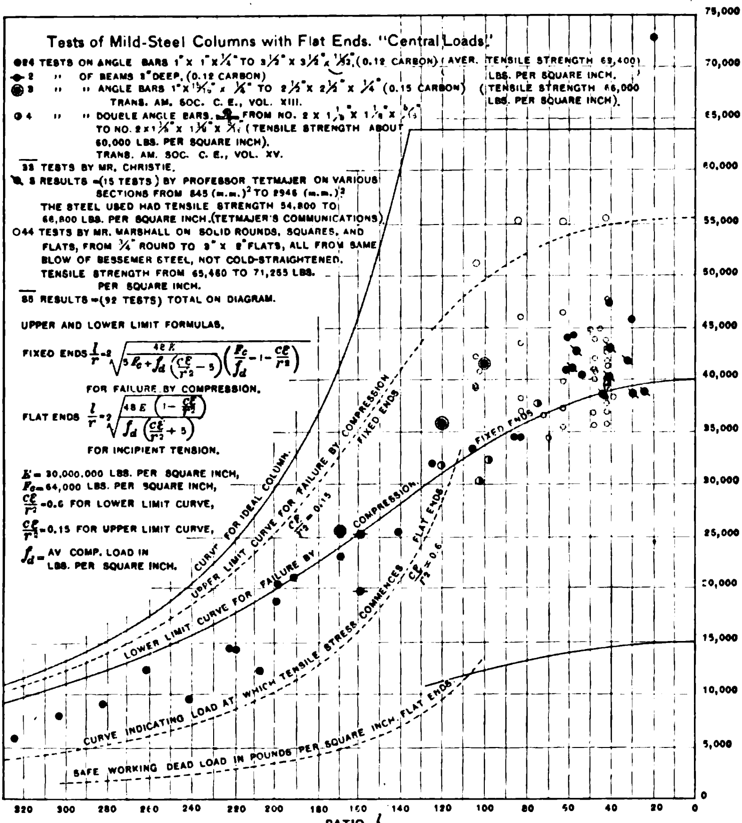


FIG. 40.

Fig. 38.—This is a combination of the diagrams on Figs. 35, 36 and 37, and shows the results of 321 tests of hinged-ended wrought-iron columns.

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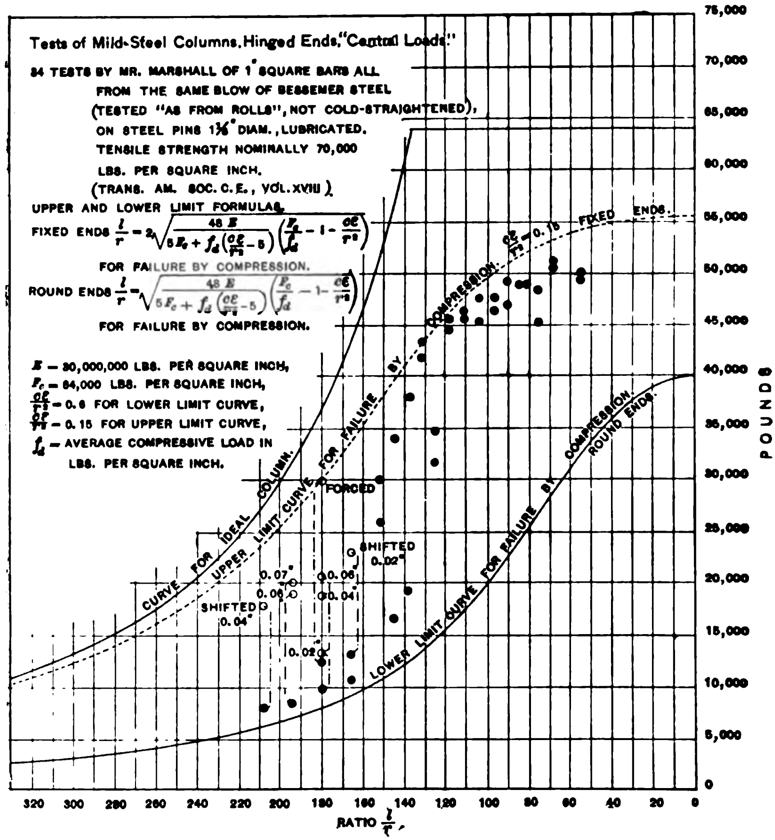
* *Transactions*, Am. Soc. C. E., Vol. xvii.

† Replotted from Plate XI of Mr. Marshall's paper, *Transactions*, Am. Soc. C. E., Vol. xvii.

Mild Steel.—Pivoted Ends.

Fig. 39.—Sixty results, representing 104 tests by Professor Tetmajer on Mild Steel Columns of various sections.* The remarks made with regard to Tetmajer's pivot-ended wrought-iron tests shown on Fig. 21, apply also to these tests on mild steel.



$$\text{FIXED ENDS } \frac{l}{r} = 2\sqrt{\frac{48E}{5F_c + f_a\left(\frac{cE}{r^2}-5\right)}\left(\frac{F_c}{f_a} - 1 - \frac{cE}{r^2}\right)}$$

$$\text{ROUND ENDS } \frac{l}{r} = \sqrt{\frac{48E}{5F_c + f_a\left(\frac{cE}{r^2}-5\right)}\left(\frac{F_c}{f_a} - 1 - \frac{cE}{r^2}\right)}$$

FIG. 41.

Mild Steel.—Flat Ends.

Fig. 40.—This diagram represents:

Twenty-nine tests by Mr. Christie;† four tests by Mr. Christie;‡ eight

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* "Tetmajer's Communications."

† *Transactions*, Am. Soc. C. E., Vol. xiii.

‡ *Transactions*, Am. Soc. C. E., Vol. xv.

results by Professor Tetmajer,* representing fifteen tests; forty-four tests by Mr. Marshall;† a total of eighty-five results, ninety-two tests.

Mild Steel.—Hinged Ends.

Fig. 41.—Thirty-four tests of hinged-ended mild-steel columns by Mr. Marshall.‡ Mr. Marshall stated in his paper that his results indicated clearly the law that:

"The elastic limit of the material is the chief factor in determining the resistance of struts of ordinary length made out of wrought iron or steel, excepting the very hardest kinds, and that the two quantities, elastic limit in compression and ultimate compressive strength are identical within a very considerable range of length of columns."

Mr. Marshall's conclusions were supported strongly by the results of his experiments, but the number of tests made was hardly sufficient to warrant the acceptance in practice of conclusions, which, if consistently applied, would allow the same stress upon a wrought-iron column having a length of 100 times the radius of gyration, as for one of only a third of that length.

The tests made by other experimenters show a considerable fall in strength with increase of length, and their evidence cannot be ignored in practice; and, judging from the results obtained by Hodgkinson, Christie, and Tetmajer, with pivot-ended columns in which the influence of frictional resistance or fixity of the column ends was eliminated, the range of length within which Mr. Marshall's law would apply, is very limited.

It will be noticed on referring to the diagrams Figs. 36 and 41, that Mr. Marshall's pin-ended tests exhibited a very rapid fall in strength in the longer column, *i. e.*, at the ratios 110 to 120 and over. This points to the probability that columns shorter than, but near, these ratios must have been in a very unstable condition, and were assisted greatly by the frictional resistance of the pin-ends.

It is also to be noted that the pin-ended tests made by Mr. Marshall on wrought iron and steel, were compared by him with the "compressive elastic limit" as determined from flat-ended and hinged-ended specimens 1 in. square by 12 ins. long,§ having a ratio $\frac{l}{r} = 41.5$, a

* "Tetmajer's Communications."

† Table No. 1 of Mr. Marshall's paper, *Transactions*, Am. Soc. C. E., Vol. xvii.

‡ Table No. 8 of Mr. Marshall's paper, *Transactions*, Am. Soc. C. E., Vol. xvii.

§ Table No. 7 of Mr. Marshall's paper.

column length quite sufficient to bring in influences which might, to a very considerable extent, affect the results, and all of these 12-in. bars deflected under their loads. In a few of the longer columns, Mr. Marshall showed how exceedingly sensitive they were to small adjustments in the testing machine, and some of these are indicated on Fig. 41. A very large increase of strength was obtained by merely moving the specimen a small fraction of an inch out of its original centering, and by the simple application of pressure at the start of a second test of a column having ratio $\frac{l}{r} = 180$, in order to cause failure by bending in the opposite direction to that in which it had first yielded, the strength was raised from 12 420 to 29 810 lbs. per square inch. This sensitiveness undoubtedly holds, in some degree, for shorter columns as well as for longer ones.

Mr. Marshall's comparison cannot therefore fairly be considered as referred to "compressive elastic limit" of the material, but only as one between the strength of certain columns, 41.5 radii of gyration long, and others of greater length.

The flat-ended tests of steel columns made by Mr. Marshall (Fig. 40) were, however, in part compared with short specimens, two sides long, from the same bars tested as columns, and it is this comparison which lends the greatest strength to Mr. Marshall's conclusions.

It is necessary to keep in view that even in a short compression specimen, eccentricity of loading will have more or less effect, and an ordinary test will only give the average stress on the material, while the important point to be determined as regards column strength is the maximum elastic fiber stress.

Mr. Marshall also made comparison between the ultimate strength of his column and the tensile elastic limit of the same bars as the columns, and the result of the comparison in most cases showed a very remarkable agreement. It is difficult, however, to see how any fixed relation of equality can exist between the elastic limit of a tension specimen and the compressive strength of a column. In a tension test, any inaccuracy of loading or lack of straightness does not tend to increase in influence, while in a compression specimen, even of moderate length, the reverse is the case.

In the curves plotted on Fig. 40 it will be noticed that no attempt is made to differentiate between the soft steel of Professor Tetmajer's

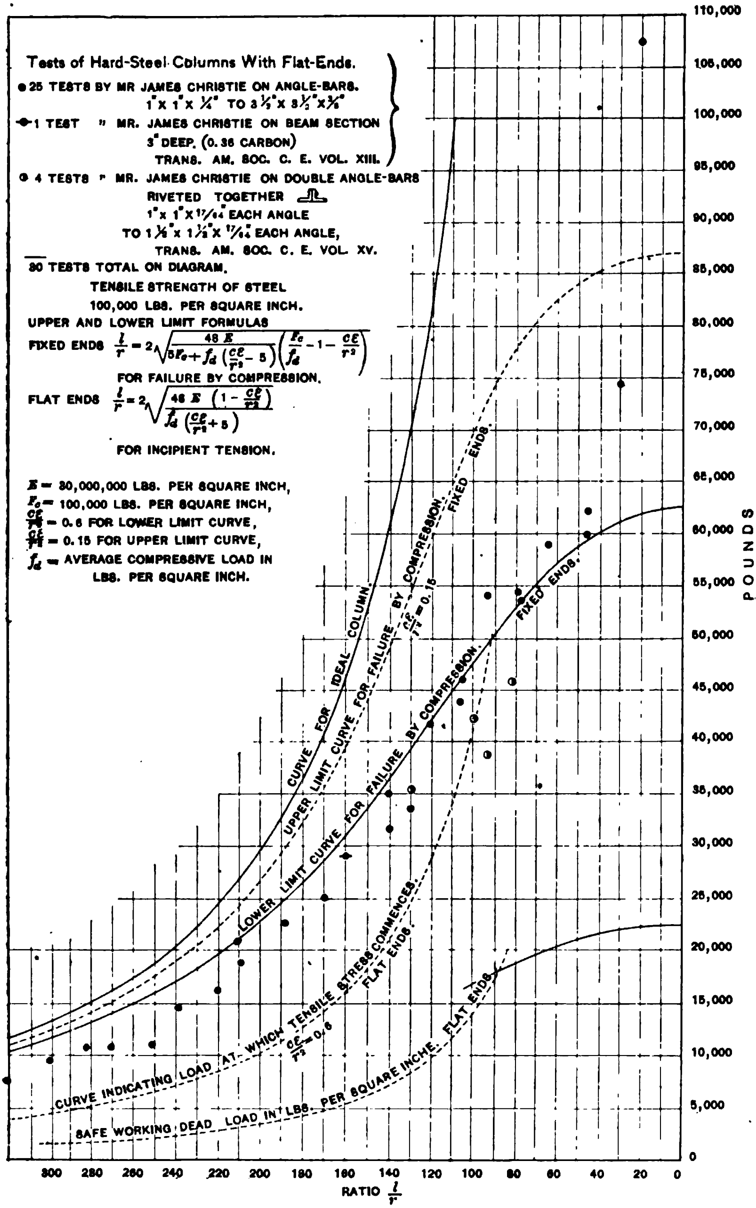

Tests of Hard-Steel Columns With Flat-Ends.
● 25 TESTS BY MR JAMES CHRISTIE ON ANGLE-BARS. 1" x 1" x ¼" TO 3½" x 3½" x ⅜"
1 TEST " MR. JAMES CHRISTIE ON BEAM SECTION 3" DEEP. (0.36 CARBON)
TRANS. AM. SOC. C. E. VOL. XIII.
4 TESTS " MR. JAMES CHRISTIE ON DOUBLE ANGLE-BARS RIVETED TOGETHER
TRANS. AM. SOC. C. E. VOL. XV.
30 TESTS TOTAL ON DIAGRAM.
TENSILE STRENGTH OF STEEL 100,000 LBS. PER SQUARE INCH.
UPPER AND LOWER LIMIT FORMULAS
FIXED ENDS
FOR FAILURE BY COMPRESSION.
FLAT ENDS
FOR INCIPIENT TENSION.
E = 30,000,000 LBS. PER SQUARE INCH,
F = 100,000 LBS. PER SQUARE INCH,
= 0.6 FOR LOWER LIMIT CURVE,
= 0.15 FOR UPPER LIMIT CURVE,
= AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION.
FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION.
FIXED ENDS.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES.
FLAT ENDS.
SAFE WORKING DEAD LOAD IN LBS. PER SQUARE INCHE.
FLAT ENDS.
POUNDS
RATIO
110,000
105,000
100,000
95,000
90,000
85,000
80,000
75,000
70,000
65,000
60,000
55,000
50,000
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0

FIG. 42.

tests and the somewhat stronger steel used by Mr. Marshall, with Mr. Christie's standing intermediate.

The number of tests in any of the three sets is not sufficient to warrant any nicety of distinction between the materials used in each, and it may be remarked that the complete range of strength, from Professor Tetmajer's minimum tensile strength of 54 800 lbs. per square inch to Mr. Marshall's tensile maximum of 71 255 lbs. per square inch, is very little greater in proportion than the range of tensile strength found in Professor Tetmajer's tensile tests of one class of wrought iron, in which the minimum and maximum tensile strengths were respectively 46 000 and 59 000 lbs. per square inch.

The curves in Figs. 39, 40 and 41, therefore, have been plotted with a value of $F_c = 64\,000$ lbs., and may be taken as applying to steel of tensile strength of from 60 000 to 70 000 lbs. per square inch.

Hard Steel.—Flat Ends.

Fig. 42.—This diagram represents:

Twenty-six tests by Mr. Christie,* and four tests by Mr. Christie,† a total of thirty tests. It may be urged with regard to this diagram that the lower-limit curve for failure by incipient tension is too low, and is not justified by the experiments.

This curve is not dependent in any way on the ultimate or elastic strength of the material, but solely on the modulus of elasticity and on the value of $\frac{c\,\varepsilon}{r^2}$, and the latter quantity has been taken the same as for all the other diagrams. The evidence of the more numerous tests on wrought iron is so strong that it does not appear advisable to count on higher loads than those indicated.

There is no reason why hard-steel struts should be assumed to be loaded with greater accuracy, or as having greater immunity from injury by cold-straightening, or as being less liable to have initial bends, than those of wrought iron or mild steel.

Timber.—Flat Ends.

Fig. 43. Tests of Yellow Pine or Pitch Pine.—The diagram represents:

Sixty-nine tests, with flat ends, made at Watertown Arsenal.‡ The

* *Transactions*, Am. Soc. C. E., Vol. xiii.

† *Transactions*, Am. Soc. C. E., Vol. xv.

‡ Exec. Doc. 12, 47th Congress, first session. These have been plotted from the figures given in Professor Lanza's "Applied Mechanics," pp. 664 to 668, 5th edition.

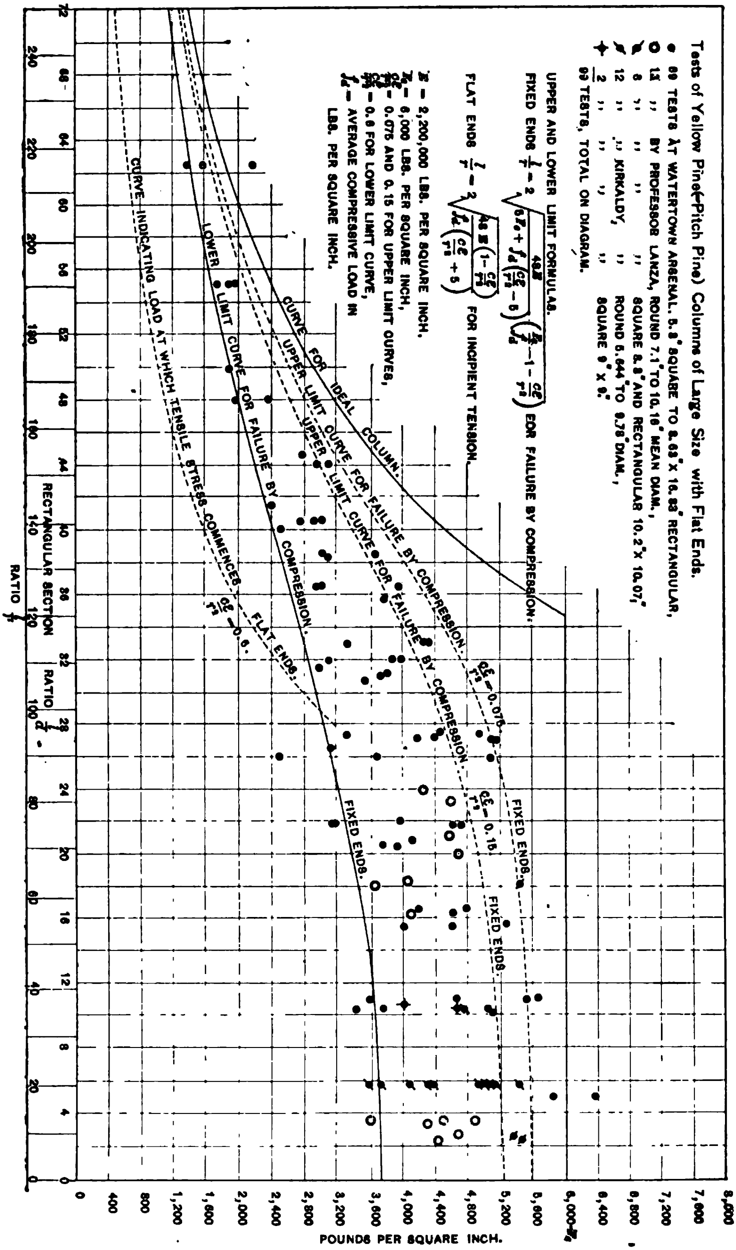

Tests of Yellow Pine (Pitch Pine) Columns of Large Size with Flat Ends.
UPPER AND LOWER LIMIT FORMULAS.
FOR FAILURE BY COMPRESSION.
FOR INCIPIENT TENSION.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES
FLAT ENDS.
FIXED ENDS.
POUNDS PER SQUARE INCH.
RECTANGULAR SECTION
RATIO

FIG. 48.

single sticks only are shown on this diagram. The built posts tested have been omitted. Three second tests of the same sticks are not plotted. Sixteen tests by Professor Lanza made at Watertown for the Boston Manufacturers' Mutual Fire Insurance Company. The flat-ended tests only are plotted.* Fourteen tests by Kirkaldy, London, England, on pitch-pine blocks with flat ends.†

Total number of tests on diagram = 99.

The inclusion of Kirkaldy's tests of pitch pine in the same diagram as the Watertown tests and Professor Lanza's tests of yellow pine is justified by a comparison of the results obtained by Kirkaldy and Lanza on short blocks. The comparison is shown in Table No. 2.

TABLE No. 2.

Kirkaldy.		Lanza.	
Round, 9.78 ins. diameter, 50 ins. long.		Round, 7.70 to 10.46 ins. diameter; Rectangular, 8.98 × 9.02 ins. And 10.20 × 10.07 ins.	Length, 23.8 to 24.38 ins.
Compressive Strength, in pounds per square inch.			
Minimum	3 586	Minimum,	3 604
Maximum	5 438	Maximum	5 452
Mean of 5 tests	4 689	Mean of 8 tests	4 658
Modulus of Elasticity.			
Average up to load of 2 400 lbs. per square inch	2 000 000 lbs.	Mean of 4 tests	1 911 385 lbs.
(*Minutes of Proceedings*, Institution of Civil Engineers, Vol. liii, p. 158.)		(Lanza's "Applied Mechanics," p. 351.)	

Fig. 44.—Sixty-six tests of rectangular flat-ended white pine columns, made at Watertown Arsenal.‡ The single sticks only are plotted on the diagram. The built posts tested have been omitted.

Fig. 45.—This diagram represents:

Thirty-six tests of square flat-ended seasoned French oak columns, by Lamandé;§ and thirteen tests of square flat-ended Dantzic oak columns, by Hodgkinson,‖ a total of forty-nine tests.

* Lanza's "Applied Mechanics," pp. 651 and 665, 5th edition.

† From Kirkaldy's Reports (Kirkaldy's Life) and *Minutes of Proceedings*, Institution of Civil Engineers, Vol. liii, p. 158.

‡ Exec. Doc. 12, 47th Congress, 1st Session. Plotted from the figures given in Lanza's "Applied Mechanics," pages 659 to 662, 5th edition.

§ Plotted from the figures given in Hurst's "Tredgold's Carpentry," page 84, 6th Edition, 1888.

‖ *Philosophical Transactions*, Royal Society, London, 1840.

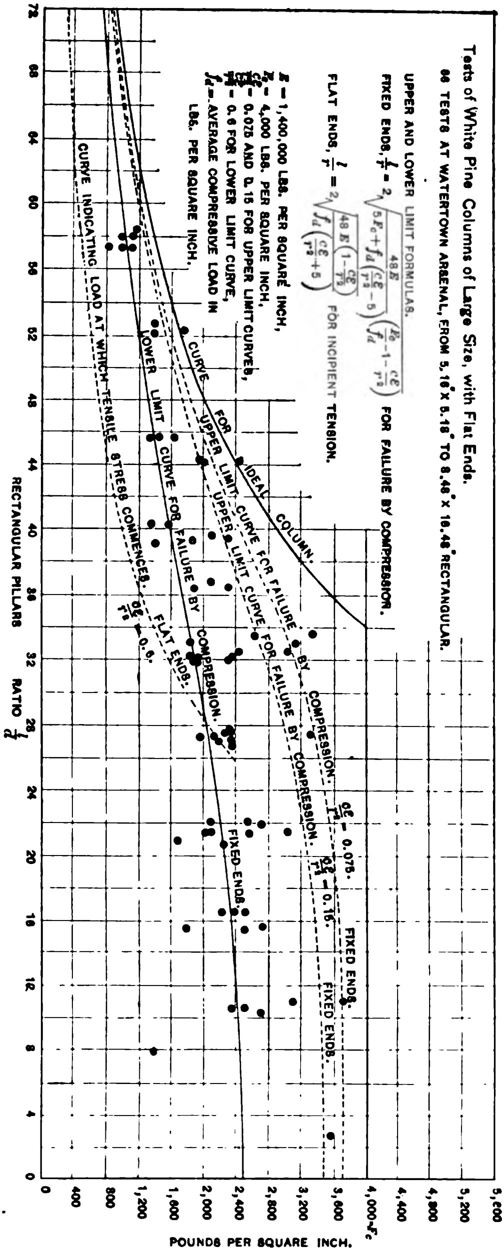

Tests of White Pine Columns of Large Size, with Flat Ends.
86 TESTS AT WATERTOWN ARSENAL, FROM 5.16" X 5.18" TO 8.48" X 16.48" RECTANGULAR.
UPPER AND LOWER LIMIT FORMULAS.
FOR FAILURE BY COMPRESSION.
FOR INCIPIENT TENSION.
E = 1,400,000 LBS. PER SQUARE INCH,
4,000 LBS. PER SQUARE INCH,
0.075 AND 0.15 FOR UPPER LIMIT CURVES,
0.6 FOR LOWER LIMIT CURVE,
AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES.
FLAT ENDS.
FIXED ENDS.
FIXED ENDS.
FIXED ENDS.
RECTANGULAR PILLARS RATIO
POUNDS PER SQUARE INCH.

FIG. 44.

In attempting to fit the curves to these three diagrams of tests of timber columns, the writer has not lost sight of the great variation known to exist in the character of specimens of timber, even from the same tree, due to a number of causes and conditions to which it is not necessary to allude. The tests plotted, however, are in each case the most important series of which the writer is aware, and they comprise the most reliable data available for the design of compression members of timber.

There have been other tests made on large-sized timbers, but they are too few in number to be of use in any endeavor to determine the influence of length on column strength. Notwithstanding the difficulties attaching to the question, it will be seen from the diagrams that the tests do, in a considerable measure, exhibit the general characteristics accompanying the numerous tests on the more uniform materials previously dealt with.

These diagrams complete the comparison attempted to be made between the results of tests of ultimate strength and the writer's rational formulas.

It may be remarked that reference has been made in the diagrams to no less than 1 620 results, representing 1 789 tests, of ultimate strength of columns of cast iron, wrought iron, steel, and timber, exclusive of the references to tests of deflection.

Application of the Formulas in Practice.

The opinion has already been expressed in the paper that the design and proportioning of columns to carry given loads should be based on the maximum fiber stresses allowable, and on the stiffness of the material, while making reference to experimental results in order to ensure having sufficient margin against ultimate failure.

In order to put this opinion into practical operation, it is necessary to adopt suitable values for the three fundamental factors in the formulas, viz., ε, F_c, and E.

It will not be necessary, in the case of "centrally" loaded columns, in practice to deal with the value of the allowable tensile stress, since tensile stress will not be developed under ordinary working loads.

The value of ε is that concerning which our knowledge must be drawn entirely from the comparisons just made with experimental re-

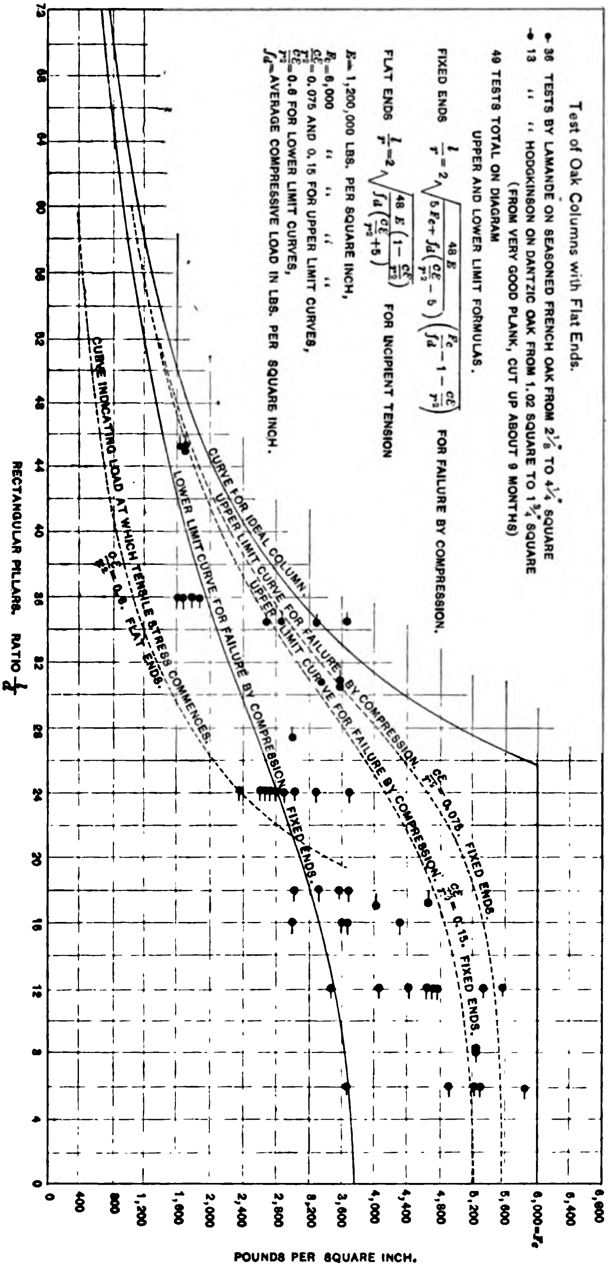

Test of Oak Columns with Flat Ends.
36 TESTS BY LAMANDÉ ON SEASONED FRENCH OAK FROM 2 1/8" TO 4 1/4" SQUARE
13 " " HODGKINSON ON DANTZIC OAK FROM 1.02 SQUARE TO 1 3/4" SQUARE
(FROM VERY GOOD PLANK, CUT UP ABOUT 9 MONTHS)
49 TESTS TOTAL ON DIAGRAM
UPPER AND LOWER LIMIT FORMULAS.
FIXED ENDS FOR FAILURE BY COMPRESSION.
FLAT ENDS FOR INCIPIENT TENSION
E = 1,200,000 LBS. PER SQUARE INCH,
Fc = 6,000 " " "
cE/r² = 0.075 AND 0.15 FOR UPPER LIMIT CURVES,
cE/r² = 0.6 FOR LOWER LIMIT CURVES,
fd = AVERAGE COMPRESSIVE LOAD IN LBS. PER SQUARE INCH.
CURVE FOR IDEAL COLUMN.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. cE/r² = 0.075. FIXED ENDS.
UPPER LIMIT CURVE FOR FAILURE BY COMPRESSION. cE/r² = 0.15. FIXED ENDS.
LOWER LIMIT CURVE FOR FAILURE BY COMPRESSION FIXED ENDS.
CURVE INDICATING LOAD AT WHICH TENSILE STRESS COMMENCES. cE/r² = 0.6. FLAT ENDS.
RECTANGULAR PILLARS. RATIO l/d
POUNDS PER SQUARE INCH.
6,000 = Fc

FIG. 45.

sults, and, although the value of $\frac{c}{r^2}$ can be at once calculated from the section of column proposed to be used, yet it appears desirable in the case of "centrally" loaded columns that one constant value should be adopted for the quantity $\frac{c\,\varepsilon}{r^2}$; and judging from experimental evidence, the value to be assigned to it should not be taken at less than 0.6, in the present state of our knowledge.

In the case of a solid cylindrical column, this value of $\frac{c\,\varepsilon}{r^2}$ corresponds to an equivalent eccentricity of only 0.3 of the radius of gyration.

The values to be given to F_c for different materials may properly be approximately those in common use for maximum working fiber stresses in solid beams under transverse stress.

With regard to the value to be given to E, it must be noted that in every column it is necessary, not only to provide that the maximum fiber stress shall be confined within proper limits, but also that the stability of the column, as governed by the stiffness of the material of which it is made, shall also have a proper and sufficient margin of safety under the loads to be imposed.

The necessity for this is at once seen by a reference to the calculations of deflection and resulting maximum fiber stresses of a long column, given on pages 15 and 16, and in connection with which it was shown that the column would be perfectly safe as regards the intensity of maximum fiber stress under a load amounting to over 90% of the ultimate supporting power.

Now, on reference to Formula (1), the general expression for the deflection of a column:

$$\varDelta = \frac{P\,l^2\,e}{8\,E\,I - 2\,P\,l^2\,y\,x} \qquad \text{(1)}$$

or its practical modification

$$\varDelta = \frac{P\,l^2\,e}{8\,E\,I - \frac{5}{6}\,P\,l^2} \qquad \text{(2)}$$

it will be seen that the deflection of any given column under any given load increases directly as the eccentricity e, and further, that if we had such a material to deal with, we need give no thought to the maximum stress developed, and we would have the curious result that the ulti-

mate load would be precisely the same whatever the amount of eccentricity might be.

The practical conclusion to be drawn from the foregoing is that the eccentricity of loading has nothing to do with the stability or instability of the column, if the condition of instability be defined as that which occurs when the deflection under load becomes infinite.

From this we see that in designing a column to carry a given load we have two totally independent modes of failure to guard against:

(1) Against failure by excessive intensity of fiber stress.

(2) Against failure by instability.

In the first we are dependent on strength, and in the second on stiffness.

We will proceed, therefore, on perfectly correct rational lines if we proportion our "centrally" loaded columns in practice to meet these two conditions, which are satisfied by using the two formulas:

$$\left.\begin{array}{r}\text{For columns}\\ \text{with both}\\ \text{ends round}\end{array}\right\} \frac{l}{r} = \sqrt{\frac{48\,E}{5\,F_c + f_d\left(\frac{c\,\varepsilon}{r^2} - 5\right)}\left(\frac{F_c}{f_d} - 1 - \frac{c\,\varepsilon}{r^2}\right)} \quad ..(7)$$

where F_c is the maximum allowable working intensity of fiber stress,

$$P = K\frac{9.6\,E\,I}{l^2}\text{ for stability}\,\ldots\ldots\ldots\ldots\ldots\ldots(2)$$

where K is a suitable fractional coefficient of safety,

or, since $$f_d = \frac{P}{a}, f_d = K\frac{9.6\,Er^2}{l^2} = \frac{9.6\,KE}{\frac{l^2}{r^2}}.$$

Applying these formulas to the material which is in most general use in constructional work at the present day, *i. e.*, mild steel, and adopting as suitable values for the various factors,

$E = 30\,000\,000$ lbs.,

$F_c = 24\,000$ lbs. per square inch, working dead stress,

$\frac{c\,\varepsilon}{r^2} = 0.6$ for columns in which $\frac{c}{r} = 2$, and $\frac{\varepsilon}{r} = 0.3$,

$K =$ coefficient of safety against instability, say $\frac{1}{3}$,

we will have the working loads per square inch on solid, cylindrical, mild-steel, round-ended columns indicated by the curves in Fig. 46, in which for every ratio $\frac{l}{r}$, the conditions are satisfied that the maximum fiber stress shall not be more than 24 000 lbs. per square inch,

and the working load shall not be more than one-third of that which will cause instability.

It will be noticed that, at about ratio $\frac{l}{r} = 88$, the two curves intersect, and at this point the factor against ultimate failure, as compared with experimental results, is somewhat less than is given either for very short or very long columns, and however rational the basis upon which the results depend, the writer thinks that, like himself, other engineers would hesitate to adopt heavier loads, in relation to ultimate

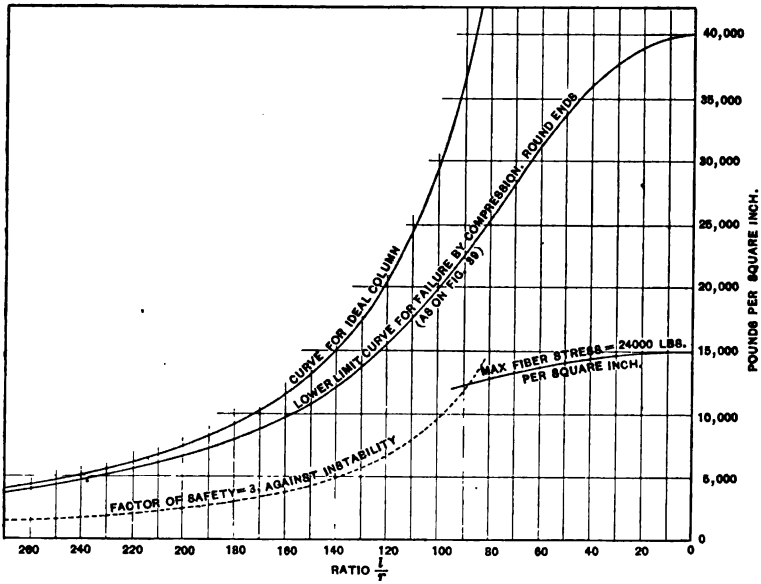
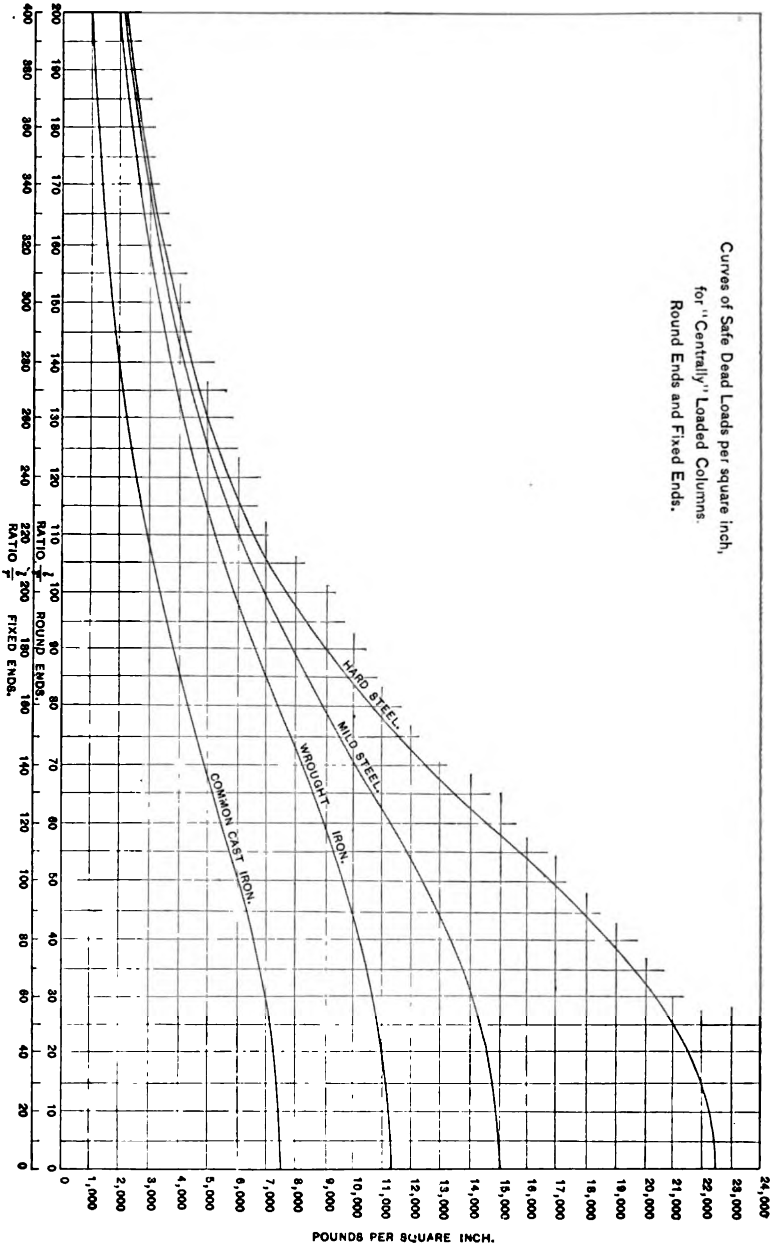


FIG. 46.

strength for a column 89 or 90 radii long, than would be considered safe for columns only one-quarter of the length, or four times the length.

Again, it may reasonably be objected that if we require the factor of safety against failure by instability of a long column to be at least 3 (assuming this as a fair value), it should follow that we would not have a much smaller value against ultimate strength as shown by experiment, in the case of columns of ratio $\frac{l}{r} = 80$ to 90.



Curves of Safe Dead Loads per square inch,
for "Centrally" Loaded Columns.
Round Ends and Fixed Ends.
HARD STEEL.
MILD STEEL.
WROUGHT IRON.
COMMON CAST IRON.
POUNDS PER SQUARE INCH.
RATIO l/r ROUND ENDS.
RATIO l/r FIXED ENDS.

FIG. 47.

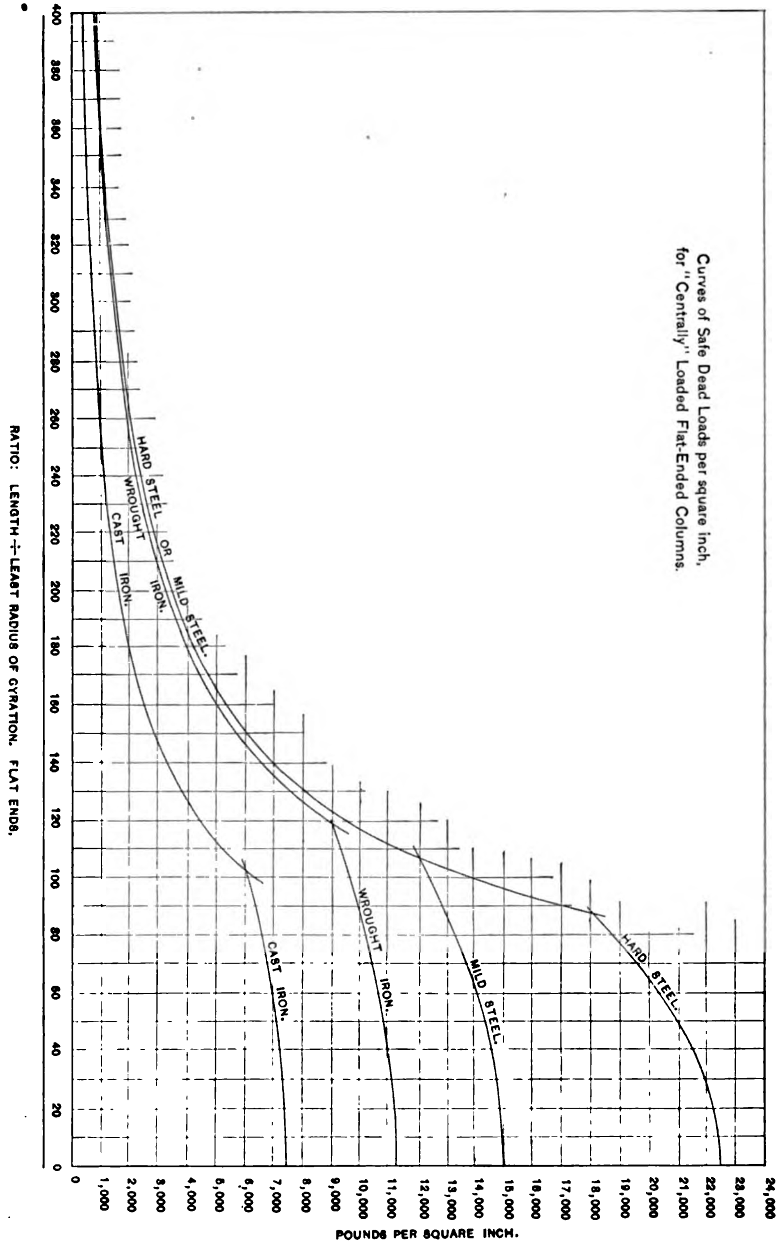

Curves of Safe Dead Loads per square inch,
for "Centrally" Loaded Flat-Ended Columns.
HARD STEEL OR MILD STEEL.
WROUGHT IRON.
CAST IRON.
CAST IRON.
WROUGHT IRON.
MILD STEEL.
HARD STEEL.
RATIO: LENGTH ÷ LEAST RADIUS OF GYRATION. FLAT ENDS.
POUNDS PER SQUARE INCH.
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11,000
12,000
13,000
14,000
15,000
16,000
17,000
18,000
19,000
20,000
21,000
22,000
23,000
24,000

FIG. 48.

TABLE No. 3.—WORKING FORMULAS FOR COLUMNS UNDER "CENTRAL LOADING."

(1)	(2)	(3)	(4)	(5)
MATERIAL OF COLUMN.	F_c = Maximum compressive fiber stress. Pounds per square inch dead load.	$\frac{KE = \text{Mod. Elas.}}{3}$ Pounds per square inch.	Value of $\frac{ce}{r^2}$	Working formulas for round-ended or pivot-ended columns. $\frac{l}{r} = \sqrt{\frac{48\,K\,E}{5\,F_c + f_d\left(\frac{ce}{r^2} - 5\right)}\left[\frac{F_c}{f_d} - 1 - \frac{ce}{r^2}\right]}$
Common cast iron..........................	12 000 = 1.2 units.	$\frac{14\ 000\ 000}{3}$ = 466.6 units.	0.6	$\frac{l}{r} = 10\sqrt{\frac{224}{6 - 4.4 f_d}\left(\frac{1.2}{f_d} - 1.6\right)}$
Wrought iron.................................. Tensile strength, 45 000 to 50 000 lbs. per square inch.	18 000 = 1.8 units.	$\frac{28\ 000\ 000}{3}$ = 933.3 units.	0.6	$\frac{l}{r} = 10\sqrt{\frac{448}{9 - 4.4 f_d}\left(\frac{1.8}{f_d} - 1.6\right)}$
Mild steel.................................... Tensile strength, 60 000 to 70 000 lbs. per square inch.	24 000 = 2.4 units.	$\frac{30\ 000\ 000}{3}$ = 1 000 units.	0.6	$\frac{l}{r} = 10\sqrt{\frac{480}{12 - 4.4 f_d}\left(\frac{2.4}{f_d} - 1.6\right)}$
Hard steel.................................... Tensile strength, about 100 000 lbs. per square inch.	36 000 = 3.6 units.	$\frac{30\ 000\ 000}{3}$ = 1 000 units.	0.6	$\frac{l}{r} = 10\sqrt{\frac{480}{18 - 4.4 f_d}\left(\frac{3.6}{f_d} - 1.6\right)}$
Timber (Unit for F_c, f_d and E = 1 000 lbs.)				
Yellow pine or pitch pine..............	2 000 = 2 units.	$\frac{2\ 200\ 000}{3}$ = 733.3 units.	0.6	$\frac{l}{r} = 10\sqrt{\frac{352}{10 - 4.4 f_d}\left(\frac{2}{f_d} - 1.6\right)}$
White pine...................................	1 800 = 1.8 units.	$\frac{1\ 400\ 000}{3}$ = 466.6 units.	0.6	$\frac{l}{r} = 10\sqrt{\frac{224}{6.5 - 4.4 f_d}\left(\frac{1.8}{f_d} - 1.6\right)}$
French oak or Dantzic oak............	2 000 = 2 units.	$\frac{1\ 200\ 000}{3}$ = 400 units.	0.6	$\frac{l}{r} = 10\sqrt{\frac{192}{10 - 4.4 f_d}\left(\frac{2}{f_d} - 1.6\right)}$

In Column (5) a unit of 10 000 lbs. is used for F_c, f_d and E for metals, and 1 000 lbs. for timber.

This objection may be met very easily and simply by using only Formula (7), with the modification of applying the factor of safety against instability to the value of E in that formula, which then becomes:

$$\frac{l}{r}=\sqrt{\frac{48\,K\,E}{5\,F_c+f_d\left(\frac{c\,\varepsilon}{r^2}-5\right)}\left[\frac{F_c}{f_d}-1-\frac{c\,\varepsilon}{r^2}\right]},$$

$$\text{where } K=\frac{1}{\text{factor of safety.}}$$

It will be seen that the actual result of this formula is to give the ratio $\frac{l}{r}$ corresponding to a given average load per square inch for material in which the maximum compressive fiber stress is limited to F_c, and for which the value of the modulus of elasticity is taken to be $K\,E$, instead of the actual value E pertaining to the material.

The use of the coefficient K in this manner has an inappreciable influence on the results of the formulas when applied to short columns, while its effect gradually increases with the length, ultimately affording a factor of safety of 3, against failure by instability, in the case of exceedingly long columns, and the factor of safety against ultimate strength is fairly even between these extremes.

Table No. 3 gives the resulting formulas reduced by the insertion of suitable values of $\frac{c\,\varepsilon}{r^2}$, F_c and $K\,E$.

For fixed-ended columns the values of $\frac{l}{r}$ obtained by the formulas in Table No. 3 are to be doubled, or in other words, for a given strength, a fixed-ended column is twice as long as a round-ended or pivot-ended one.

For flat-ended columns, the relation between the strength and the ratio $\frac{l}{r}$ is the same as for fixed-ended columns, up to the point when incipient tension is the controlling factor, and it is then necessary to use the modified formulas as previously described for that condition, except that, as we are now dealing with safe working loads, the coefficient of safety K must be inserted in Formula (9), which then becomes

$$\frac{l}{r}=2\sqrt{\frac{48\,K\,E\left(1-\frac{c\,\varepsilon}{r^2}\right)}{f_d\left(\frac{c\,\varepsilon}{r^2}+5\right)}}$$

or inserting

$$K = \tfrac{1}{3} \text{ and } \frac{c\,\varepsilon}{r^2} = 0.6,$$

$$\frac{l}{r} = 2.138\sqrt{\frac{E}{f_d}}$$

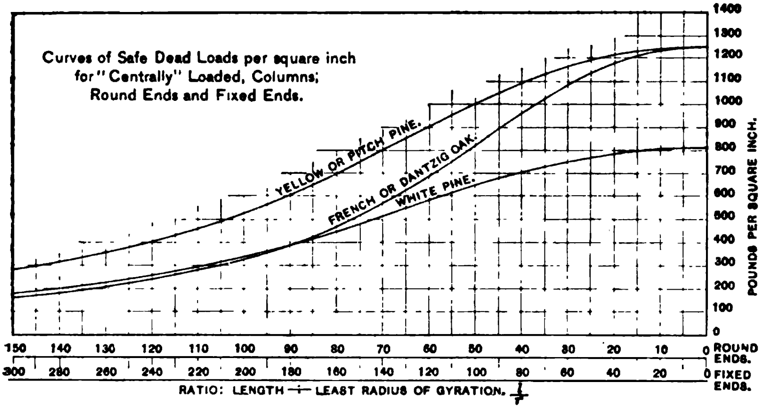


FIG. 49.

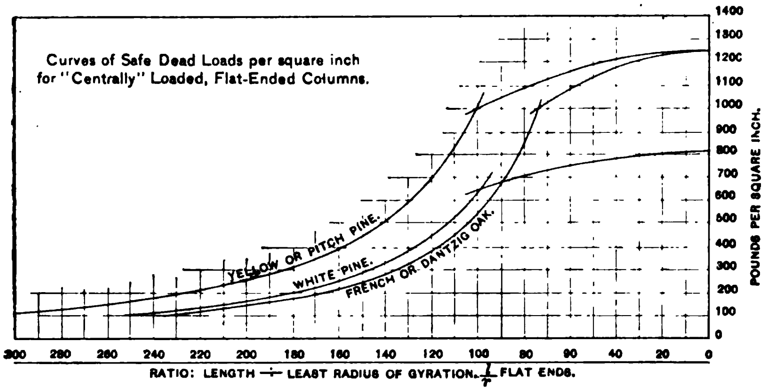


FIG. 50.

The results of the various reduced formulas for working dead loads in Table No. 3 for round-ended columns, and of the modifications above stated for fixed and flat-ended columns, are shown on various diagrams accompanying the paper, and are also shown in Figs. 47, 48, 49 and 50, drawn to a larger vertical scale for convenience in use.

With regard to the curves for working dead-loads on fixed-ended and flat-ended columns, it must be remembered that in a laboratory or test-room experiment on a fixed or flat-ended column, the conditions of end fixing are totally different from those met in ordinary practice. The rigidity of the testing machine, bearing faces, as compared with the column under test, is far greater than is afforded by any end bearing or connection in practice, and for this reason it is well not to count, in actual work, upon such high strengths as are indicated by experiment.

The only modification necessary in dealing with live loads or moving loads instead of the dead loads for which the stresses adopted in Table No. 3 are suitable, consists obviously of a reduction in the value of F_c or F_t, and an increase in the value of K, although in the writer's practice he adopts the method of increasing the moving load by a suitable percentage, dependent on the character of the load, and then treats the result as a dead-load equivalent.

Columns Under Intentionally Eccentric Loads.

The experimental data, to which reference can be made in regard to columns on which the load is imposed with large eccentricity as compared with the size of column, are exceedingly few, and as far as the writer is aware, are limited to the tests by Tetmajer (Figs. 24 and 25); but as the theoretical principles upon which the formulas are based are those in common use and acceptance in all cases involving transverse bending simply combined with the influence of direct loading, there can be no more hesitation in applying them to columns than there is in any case of simple bending.

In making use of the formulas in practice, it is, however, not only sufficient to take into account the actual measured eccentricity and the value of $\frac{c}{r^2}$ as fixed by the section, but allowance should be made for the "accidental" value of $\frac{c\,\varepsilon}{r^2}$, as dealt with in the case of presumably central loading.

This being the case, it will be necessary to add the value of $\frac{c\,\varepsilon}{r^2} = 0.6$, as determined for centrally loaded columns, to the measured

value of $\frac{c\,e}{r^2}$, as obtained from the intended eccentricity e, and the dimensions and form of the sections which fix the value of $\frac{c}{r^2}$.

In this way a perfectly rational recognition is given to the form of section in precisely the same manner as in the case of beams under simple flexure, and the formulas for the relation between the ratio $\frac{l}{r}$ and the average load per square inch for a free-ended column carrying an eccentric load thus become, for safe working loads,

$$\frac{l}{r} = \sqrt{\frac{48\,K\,E}{5\,F_c + f_d\left\{\left(\frac{c\,e}{r^2} + 0.6\right) + 5\right\}}\left[\frac{F_t}{f_d} - 1 - \left(\frac{c\,e}{r^2} + 0.6\right)\right]}$$

where F_c is the maximum allowable working fiber stress in compression, or

$$\frac{l}{r} = \sqrt{\frac{48\,K\,E}{5\,F_t - f_d\left\{\left(\frac{c\,e}{r^2} + 0.6\right) + 5\right\}}\left[\frac{F_t}{f_d} - 1 + \left(\frac{c\,e}{r^2} + 0.6\right)\right]}$$

where F_t is the maximum allowable tensile fiber stress (and being tensile it must be accorded the minus sign independently of the fixed signs in the formula, as has been pointed out previously).

These formulas again reduce to very simple expressions when the values of $K\,E$, F_c or F_t and $\frac{c\,e}{r^2}$ are inserted, as will be seen by the following application to mild steel, using the same values for the various quantities as adopted for "centrally" loaded columns.

Since the maximum compressive stress actually developed in a column of symmetrical section always exceeds the maximum tensile stress, it will not be necessary, in the case of mild steel, to use the formula containing the value of maximum allowable tensile stress F_t, as this may be taken, for this material under working loads, as having the same value accorded to F_c, the allowable compressive fiber stress.

In the case of cast iron it will, of course, be necessary to use both expressions, and to adopt whichever gives the lower value to the ratio $\frac{l}{r}$ for any given load f_d.

The formula, then, for the following values of $\left(\frac{ce}{r^2} + 0.6\right)$ inserted will be for mild-steel columns of symmetrical section, carrying dead loads, using 10 000 lbs. as a unit for E, F_c and f_d:

$$\left(\frac{ce}{r^2} + 0.6\right) = 2 \frac{l}{r} = 10 \sqrt{\frac{480}{12 - 3f_d}\left(\frac{2.4}{f_d} - 3\right)}$$

$$\text{“} \quad \text{“} \quad = 4 \frac{l}{r} = 10 \sqrt{\frac{480}{12 - f_d}\left(\frac{2.4}{f_d} - 5\right)}$$

$$\text{“} \quad \text{“} \quad = 6 \frac{l}{r} = 10 \sqrt{\frac{480}{12 + f_d}\left(\frac{2.4}{f_d} - 7\right)}$$

$$\text{“} \quad \text{“} \quad = 8 \frac{l}{r} = 10 \sqrt{\frac{480}{12 + 3f_d}\left(\frac{2.4}{f_d} - 9\right)}$$

The working out of these values of the formula is shown by the curves on Fig. 51.

A reference to the remarks previously made in connection with Fig. 24 will show that in dealing with columns of unsymmetrical section it may be necessary to consider the question of tensile stress, and to use the modification of the formula suitable to such conditions.

Columns Subjected to Side Loads.

The effect of a side load in addition to the end load on a column is easily traced by the aid of the principles and formulas in the foregoing pages. To illustrate this it will be convenient to take a direct example.

Assume a mild-steel column with pivot ends, and having the following particulars:

$l = 100$ ins. = length,

$I = 10$ ins.^{4}

$r = 1$ in.

$c = 2.5$ ins., from axis to extreme fibers,

Sectional area = 10 sq. ins.,

$E = 30\,000\,000$ lbs.;

and assume the side load to be 4 000 lbs., distributed along the column's length.

This will produce a bending moment at the center,

$$M = \frac{4\,000 \text{ lbs.} \times 100 \text{ ins.}}{8} = 50\,000 \text{ inch-pounds},$$

and the resulting maximum fiber stress will be

$$f = \frac{Mc}{I} = \frac{50\,000 \times 2.5}{10} = \pm\ 12\,500 \text{ lbs. per square inch.}$$

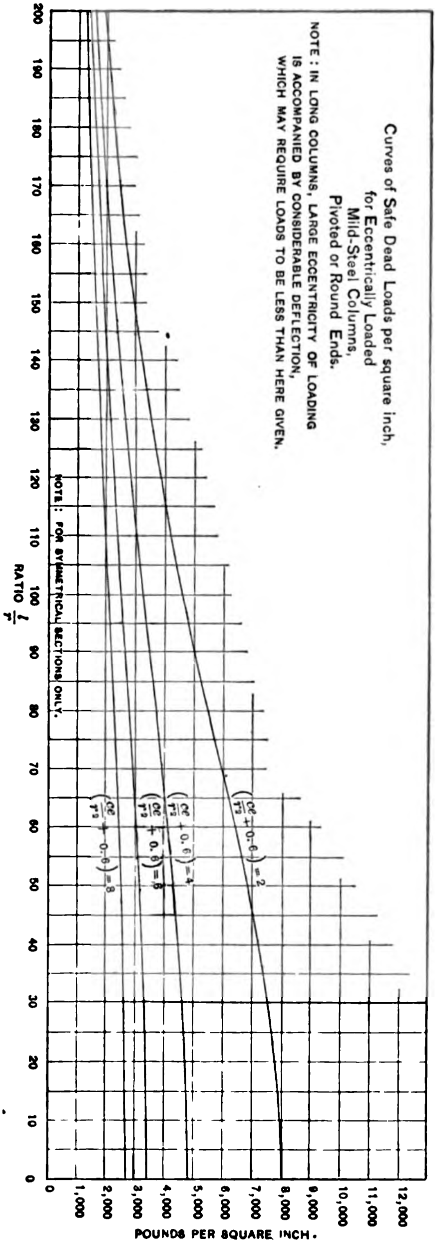

Curves of Safe Dead Loads per square inch,
for Eccentrically Loaded
Mild-Steel Columns,
Pivoted or Round Ends.
NOTE: IN LONG COLUMNS, LARGE ECCENTRICITY OF LOADING
IS ACCOMPANIED BY CONSIDERABLE DEFLECTION,
WHICH MAY REQUIRE LOADS TO BE LESS THAN HERE GIVEN.
NOTE: FOR SYMMETRICAL SECTIONS ONLY.
$\left(\frac{ce}{r^2}+0.6\right)=2$
$\left(\frac{ce}{r^2}+0.6\right)=4$
$\left(\frac{ce}{r^2}+0.6\right)=6$
$\left(\frac{ce}{r^2}+0.6\right)=8$
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
RATIO $\frac{l}{r}$
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11,000
12,000
POUNDS PER SQUARE INCH.

FIG. 51.

The side-load bending moment diagram will have a parabolic outline, as shown in Fig. 52, the central depth being 50 000 inch-pounds to any convenient scale.

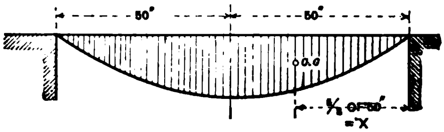


FIG. 52.

Treating each half of the column (or beam) as a cantilever, with respect to its central section, the deflection at the center will be

$$v = \frac{A\,X}{E\,I},$$

where $A = 50\,000 \times 50$ ins. $\times \frac{2}{3}$, and $X = \frac{5}{8}$ of 50 ins., and therefore,

$$v = \frac{(50\,000 \times 50 \times \frac{2}{3}) \times \frac{5}{8} \text{ of } 50)}{30\,000\,000 \times 10} = 0.1736 \text{ in.}$$

In order to determine the deflection of the column under end loads it is now only necessary to treat it in precisely the same way as already indicated for a column having an initial curvature, with the value of central versed sine, $v = 0.1736$ in., keeping in view, when considering its strength as against end loads, that we have already absorbed 12 500 lbs. of the allowable maximum fiber stress.

In the present case it is not difficult to determine the precise theoretic form of the curve assumed by the bent column under its side load alone, this curve depending on the disposition of the load.

In practical work, however, it is not necessary to go to this degree of accuracy, and we may assume the curve to be a parabola, without involving any serious error, and the justification is found in the fact that had we assumed the side load to be concentrated at the center, the assumption of a parabola instead of the precise curve taken by the bent column would only involve an excess, as regards its influence on the deflection, of a little over 4 per cent. As compared with the curve produced by a distributed load, the parabola would have a much less difference than even that named.

In order to make the comparison between the column with side load, and one of ordinary character with presumably central loading, it will be necessary to treat both on the same lines as far as possible, and in calculating the safe dead load on the centrally loaded column, we have limited the maximum fiber stress to 24 000 lbs. per square inch, and have at the same time used a value of

$$K E = \frac{30\ 000\ 000}{3} = 10\ 000\ 000 \text{ lbs.},$$

and also a value of $\frac{c\ \varepsilon}{r^2} = 0.6$.

These same values must be used also in the side-loaded columns as regards the influence of the end loads at least.

The value of $\frac{c\ \varepsilon}{r^2} = 0.6$, in the case of this column with a radius of gyration $= 1$ in., corresponds to an equivalent eccentricity of 0.24 in., and we thus have a column with an initial curvature measured by a versed sine of $v = 0.174$ in. and an eccentricity of loading $\varepsilon =$ 0.24 in., and making use of Formula (3) (with K inserted) to determine the deflection caused by different values of the end load, P, we have

$$\Delta = \frac{P\ l^2\ (\varepsilon + 2\ y_1\ x_1\ v)}{8\ (K\ E)\ I - 2\ P\ l^2\ y\ x}.$$

We have already assumed the curve of the bent column, from both side and end loads, to be a parabola, and the values of $2\ y_1\ x_1$ and $2\ y\ x$ will each be $\left(2 \times \frac{2}{3} \times \frac{5}{8}\right) = \frac{5}{6}$, and, inserting these and the values of the other factors in the formula, we have:

$$\Delta = \frac{P \times 10\ 000\ \left(0.24 + \overline{\frac{5}{6} \times 0.174}\right)}{(8 \times 10\ 000\ 000 \times 10) - \left(\frac{5}{6}\ P \times 1\ 000\right)} = \frac{0.462\ P}{96\ 000 - P} \text{ ins.},$$

for the elastic deflection of this column under any end load P.

TABLE No. 4.

P, in pounds.	$\Delta = \frac{0.462\ P}{96\ 000 - P}$	Bending moment from end loads. $M = P\ (0.414 + \Delta)$ inch-pounds.	Maximum fiber stress from end-load bending moments. Pounds per square inch.	Direct stress. $f_d = \frac{P}{a}$. Pounds per square inch.	Maximum fiber stress from side-load bending moments. Pounds per square inch.	Total maximum compressive fiber stress. Pounds per square inch.
30 000	0.21	18 720	4 680	3 000	12 500	20 180
32 000	0.231	20 640	5 160	3 200	12 500	20 860
34 000	0.253	22 680	5 670	3 400	12 500	21 570
36 000	0.277	24 880	6 220	3 600	12 500	22 320
38 000	0.303	27 240	6 810	3 800	12 500	23 110
40 000	0.33	29 760	7 440	4 000	12 500	23 940

The bending moment at the center of the column, from end loads alone, will be $M = P\,(\varepsilon + v + \varDelta) = P\,(0.24 + 0.174 + \varDelta)$

$$= P\,(0.414 + \varDelta) \text{ ins.};$$

and now calculating the deflections, bending moments, and resulting maximum fiber stresses caused by the end loads, and combining them with the maximum fiber stress of 12 500 lbs. caused by the end loads alone, we have the results given in Table No. 4.

From these results we see that with an end load of 4 000 lbs. per square inch, the maximum fiber stress reaches the limiting value of 24 000 lbs. per square inch, and, on referring to Fig. 47, we find that with the presumably centrally-loaded column, under the same basis of calculation, the safe load f_a would be (for the ratio $\frac{l}{r} = 100$) 7 000 lbs. per square inch, or in other words, the side-load has reduced the safe end-load by nearly 43 per cent.

The foregoing calculations will serve to illustrate the proper mode of treating the conditions met with in side-loaded columns, although they have been applied to a column of such proportions as would rarely be trusted to carry any appreciable amount of side-load other than its own weight.

The values of I, r and c, and the sectional area assumed, are roughly those of a rolled girder section 12 ins. × 5 ins. × 33 lbs. per foot.

The formulas were reduced by the writer in 1896 and 1897, for use in his own practice, and he hopes that the results given in this paper may be found to be fairly in accordance with the leading principles set forth in the opening sentences as being those with which a column theory and its resulting formulas should comply, at least from the point of view of the practicing engineer rather than that of the pure mathematician.

The question of the proper construction of columns so as to ensure the full development of the strength which may rightfully be expected from any given section, is one which has not been dealt with herein, although it is certainly of the greatest importance, in view of the inferior types sometimes adopted in present-day practice.

Of these inferior types the writer would especially draw attention to one which appears to be in more common use, and that is, the column with batten-plate connections between the main members forming the column, as a substitute for a properly constructed web,

and a reference to the illustrations of the failure of a bridge in Servia,* will give ample evidence of the inefficiency of the type. Its inferiority cannot be questioned in the case of columns carrying eccentric loads, or having to resist shearing stresses from side wind-loads as in some of the lofty buildings constructed in America.

The writer is conscious that in these pages there will be found (and necessarily so) many points of resemblance and identity, both in views and in their expression, with the contributions of others, and he desires to acknowledge his indebtedness to the numerous writers from whose papers he has drawn information and assistance.

While, from a strictly mathematical point of view, objections may possibly be raised to the reasoning and results in this paper, yet if a close comparison be made with more highly analytical investigations, it is believed that the differences from the latter will be found to be of little importance in any practical sense.

The writer was impelled to attempt the deduction of new formulas by having to design a number of columns to carry heavy loads with exceptionally large eccentricity, for which condition he could find no theory or formulas sufficiently simple and easy of application under varied circumstances.

The concentration of such a large mass of scattered experimental evidence will, it is hoped, be of sufficient value to justify the labor involved in its collection, examination and arrangement in diagram form.

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* *Engineering*, February 3d, 1898.

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INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

RIVER HYDRAULICS.

Discussion.*

By L. J. Messrs. LE CONTE and JAMES A. SEDDON.

Mr. Le Conte. L. J. LE CONTE, M. Am. Soc. C. E. (by letter).—It has been generally known that there exists a large amount of valuable information on the files of the several district offices in the Mississippi Valley which, from lack of funds, has never been properly studied and digested by any competent person. The author has undertaken this task, and has presented many facts and important conclusions.

He brings out strongly the important feature that local slope is of no importance in the formulas for stream flow, because it is an unstable as well as a secondary result of local changes in cross-section; and since the cross-sections themselves are constantly changing, the local slope also changes correspondingly, and yet the discharge remains constant all along the entire reach. These potent facts show how carcful an engineer should be in selecting the upper and lower limits of the characteristic reach, in order to ascertain, for the entire reach, the true average fall, the true mean characteristic depth, and the corresponding constant discharge. That is to say, he must not allow himself to be misled by any non-characteristic local slope or cross-section, their lack of stability being in itself sufficient evidence of their secondary character. As an extreme case, showing the lack of influence of surface-slope, we have at Donaldsonville an actual

*Continued from February, 1900, *Proceedings*. See October, 1899, *Proceedings*, for paper by James A. Seddon, M. Am. Soc. C. E., on this subject. Under the rules for publication, additional discussion on this paper received prior to April 27th, 1900, will be published with the paper in the next volume of *Transactions*.

reverse-slope extending up the river for 20 miles or more, and yet the great quantity of water moves down the river with the same discharge as where the slope is down stream. Mr. Le Conte.

The author's system of triple gauges at each station is certainly the most rational device, and, at the same time, gives the best and quickest results. The amount of useful information which can be obtained by means of such simple apparatus is remarkable.

The author's comments on levee effects, in always lowering the low-water plane and thus facilitating flood-water propagations down the river, will be welcome news to river engineers as well as riparian land owners. It is a matter of paramount importance in reclamation works, and should command the closest attention. The whole story is shown in an indisputable form by a study of the discharge scales for the past ten years.

The author's statement of the facts pertaining to flood movement and his mathematical deductions therefrom are certainly interesting and valuable. The full value of their bearing upon the actual flattening out of the flood waves and the correspondingly rapid flood-propagation down the river channel are matters of great importance when dealing with large rivers, and the writer thinks, that at least in the lower divisions, even of rivers of ordinary size, the influence of this flood-pulse has been largely underestimated. The author emphasizes the fact that in the Lower Mississippi the prodigious flood-movement cannot be explained by the direct flow of the waters down stream, because the flood-wave crest travels down the river four times as fast as the water. This feature, together with the mathematical deductions, shows unmistakably that a close relation exists between these flood-propagation phenomena and the laws governing tidal propagation in shallow estuaries and tidal rivers; the only difference being that the flood-pulse originates in the upper river and is propagated down stream, while the ordinary tidal-pulse originates in the ocean and is propagated up stream against the flow of the current.

It is to be hoped that the author will continue to give us information in this line and thus enable us to weed out the weak points in existing standard formulas based on experiments of small scale.

James A. Seddon, M. Am. Soc. C. E. (by letter).—From the limited discussion offered, the writer has come to question whether he really has succeeded in making the hydraulic system, presented in his paper, as clear as he thought he had. As the field of river data has in it certainly a good deal of brush wood, in which even the trained investigator may lose himself utterly for a time, and which probably deters many from ever entering it, the writer has concluded that he can best serve the purposes of a discussion by presenting briefly the outline of his system in its application to a simpler case, and one with which engineers are more generally familiar. Mr. Seddon.

Mr. Seddon. Assuming for this a flume, say 5 ft. wide, set on some uniform grade, which it is not now necessary to consider; and taking 10 000 ft. of it as the reach through which observations are to be made; automatic gauges with zeros at the bottom of the flume set at the upper and lower end of it; the water run out and the whole thing ready for the experiment.

The flow then would be started, very little at first, increasing steadily until the flume was running full for a time, and then gradually shut off and the water again drained out of it. During this time the records of the two gauges would be the only data required to determine the discharge of the flume at any and every level and through all these variations of slope found in its filling and emptying.

For instance, each automatic gauge would trace the changing level to a common time scale, and should, of course, be set accurately to give this from the bottom of the flume, enlarged if necessary, and so proportioned to the time scale as to mark the coincident time and level most distinctly. Then, say, from the trace of the rising level at each gauge, the data of Columns (1), (2) and (3) in Table No. 2 are taken.

TABLE No. 2.

Water level in flume h, in feet.	TIME AT WHICH LEVEL IS REACHED ON GAUGES.		Difference of time, in seconds. $= \Delta T$.	$\frac{10\,000}{\Delta T} = m$.	$m \times 5 = m W$.	Discharge in cubic feet per second $= Q$.
	Upper gauge.	Lower gauge.				
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0.1	6 00 00 A.M.	8 34 20 A.M.	9 260	1.08	5.4	0.5
0.3	6 40 00 "	8 54 30 "	8 070	1.24	6.2	1.7
0.5	7 20 00 "	9 19 00 "	7 140	1.40	7.0	3.0
1.0	8 10 00 "	9 42 40 "	5 560	1.80	9.0	7.0
1.5	9 00 00 "	10 15 50 "	4 550	2.20	11.0	12.0
2.0	10 00 00 "	11 04 10 "	3 850	2.60	13.0	18.0
3.0	12 00 00 M.	12 49 00 P.M.	2 940	3.40	17.0	33.0
4.0	2 00 00 P.M.	2 39 40 "	2 380	4.20	21.0	52.0
5.0	4 00 00 "	4 33 20 "	2 000	5.00	25.0	75.0

From this, Columns (4), (5) and (6) are readily computed; and, platting these values of $m\ W$ to h gives the line shown on Fig. 18, which is expressed by the empiric equation $m\ W = 5 + 4\ h$.

Then from Equation II,

$$\frac{d\,Q}{d\,h} = m\ W = 5 + 4\ h,$$

and integrating,

$$Q = 5\ h + 2\ h^2 \pm c.$$

In the more general case, where h is the stage scale and is taken from a zero at an arbitrary low water, c, the constant of integration, is simply the regular discharge of the river at that level, upon which

the variations of discharge given by the flood movement are imposed. Mr. Seddon. But here, where h is taken from the bottom of the flume, and the values of Q and h become zero together, c is then zero, and the complete discharge equation is $Q = 5\,h + 2\,h^2$, from which the values of Q given in Column (7) are computed, and platted on Fig. 18 show the discharge curve as determined.

This, of course, is merely an illustration, and may not correspond with any actual case; and, indeed, no case is actually fixed until the grade of the flume is given, which has not yet been even considered. It simply presents the process of determining, in a channel of given dimensions, a whole discharge sequence at once merely with the records of two gauges.

Neither does the writer claim that it is a better measure of the discharge here than the ordinary method of observing the velocity at a number of points in the section and from these computing the flow; or the alternative of running a steady flow into a basin of a given size, and from the filling in the period calculating the discharge. Considering the wide range of discharge values which the proposed method covers in a single operation, with no meters to rate or observers' errors to question, and the whole thing reduced with little more work than that of a single discharge observation, if the writer had any experiments on the flow of conduits or flumes to make, he would certainly try it. But, until it is tried, he is not ready to consider its precision. All this rests upon the precision with which the time element involved in m may be determined, and is a matter yet to be tested.

But this is not an untried field in the writer's studies of rivers. He has shown that here in favorable cases m may be determined within limits hardly exceeding 1%, and even on the Missouri probably close to 3 per cent. This has been done also simply with the ordinary gauge data, and how unsuitable this is for such determinations should be understood. In the first place, the gauge observers are local men employed in other occupations and paid a small sum monthly to take the readings and send them to the different offices, and there is no guarantee that they do not generally read their gauges earlier or later than the set time by an hour or so, as suits their convenience. Again, the water surface is frequently quite rough, and even the most careful man may make a mistake of a tenth or more in estimating its level, with waves running a foot high. And, finally, the gauges themselves are often set with no nice regard for any real precision in their readings, taking in the pulse of an eddy, or the extreme variations of slope through a bridge span, and in the case of cable gauges, all the temperature errors incident to measuring down from the lower cord of a bridge to the water surface with a weight at the end of a wire.

The precision attained in the determination of m, with such data,

Mr. Seddon. leaves no doubt whatever that surface observations, planned and taken with that specially in view, would bring this proposed measure of discharge into a field of more reliable and exact data than the best that has yet been taken, not to speak of the whole mass collected, which now requires years of study to sift in order to form some idea of what is reliable in it.

This, however, is simply the case where the dimensions of the channel are given. In tidal rivers, for the determination of their peculiar form of discharge curves, with a zero at both the upper and lower levels joined by plus and minus branches, probably this process would be all that was wanted. But in other rivers it may be desirable to first calculate dimensions; and going back to the flume to consider such a case it is plain that the discharge curve of Fig. 18 is not the curve for the head of the reach nor the foot of it, but for some intermediate average; and that this again changes with the grade of the flume and different rates of filling and emptying.

The only constant in the matter with a set grade is the intermediate value of discharge between equal rates of rise and fall, or the uniform flow at the different levels, and this corresponds to the mean of the rising and falling flood movements. In the case of the flume, very possibly, accurate dimensions might be determined with simply rising discharges observed in the middle of the reach and the coincident flood movements through it; but in the river, where a general stage scale for the reach is also to be gotten, the process of standard curves and mean m' is preferable.

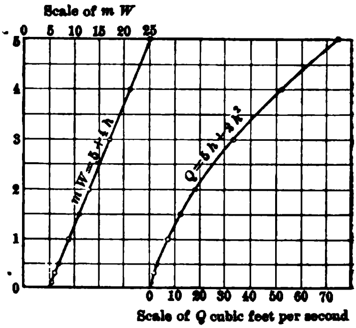

FIG. 18.

However, the dimensions once determined in this way, if the form of the river is fixed, W at any time in the equation $\frac{d\,Q}{d\,h} = m\,W$ is given, and simply measuring m then will settle all questions of what effects the larger slope on the rise and the less on the fall may have in that reach. So far as his own studies go with these rivers of fixed regimen, the writer would say that one reach may show these effects and the next may not; and that this depends on whether a general slope controls the flow there or special sections. But it is certainly a field that needs further observations.

In a general way, also, the same may be said of alluvial rivers. But since the writer in 1885 turned a given flow down an inclined plane covered with river sand, and found that it came to about the same

velocity no matter what inclination he gave it, he has given up trying to explain all the phenomena of alluvial river flow with velocity-slope relations. And, indeed, he thinks it is well here for every investigator to first try and see broadly what slope does not explain before he puts any great amount of time into fitting local data thereto. Mr. Seddon.

Take a case from the Arkansas City, 1884–85, discharges illustrating it with Fig. 19, in which scales are ignored for the sake of bringing the matter out clearly. It is seen that this change of plane, or different levels of the same discharge in the first and second periods, comes in between Helena and Arkansas City and from there on is continuous to Vicksburg. If the different slopes on the rise and fall are taken to explain it from Arkansas City down, it must at the same time be explained how this flood with about the same slope differences shows no such marked change in the discharge levels from Cairo to Helena. And, further, in the same connection, while the writer does not draw conclusions from the data of single sections, it may yet be noted, that Mr. Starling's coefficients* for this Arkansas City section, with these slope differences taken into account, show an even more marked difference between the two periods than that given by the discharges.

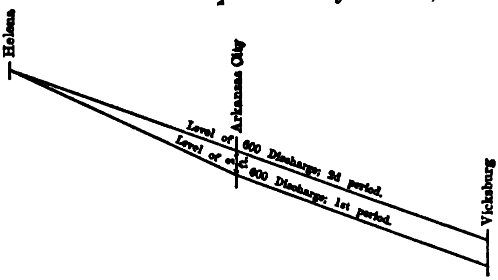

FIG. 19.

The writer calls this a change of plane, and, following the flood from Cairo down, as he finds no general difference of any such magnitude that he can assign to the varying slopes of the rising and falling river, he does not recognize them as the cause of it. He does not mean that such effects are not there, but simply that they are not in this proportion. But these are not the only changes of slope involved in the matter. In Fig. 19 a change in the slope of the discharge levels from Helena is shown, and it has been stated that this corresponds roughly with a flood out of the Arkansas. It should also be noted that in such a period when the discharge is 600 at Helena it may be 700 to 800 at Arkansas City; and this change in the discharge level is then but a fraction of the actual change in the surface slopes leading down to it. Had Mr. Starling, in place of taking his slopes from Arkansas City to Greenville, taken them from above to the mouth of White River, where this change really comes in, he would have found a much better ground on which to base these discharge differences.

* "The Discharge of the Mississippi River," *Transactions*, Am. Soc. C. E., Vol. xxxiv, Fig. 20.

Mr. Seddon. But why, when such a change has once come into the river, it seems to hold for a considerable time after the tributary flood has run out, and is continuous, apparently without change in its magnitude, for hundreds of miles down, are matters which the slopes do not explain. However, to answer Mr. Todd's question of what the writer's system of corrections is: He will have to call them, for the present, simply accumulated experience of where and under what conditions such changes are likely to come in, and their probable magnitudes. He does not feel that he has quite gotten to the bottom of this phenomenon, and is not ready to say that any single cause, that is altogether satisfactory, can be assigned to it.

Going back, however, to the flume for comparison, it may be seen that while the general grade in the alluvial river is also fixed; it is still the case where at any location the bottom, or the zero of discharge, from time to time may be shifted. Simply shifting the discharge curve as a whole up or down, the writer has found covers at least the greater part of the phenomena; and as this is done in a minute, and records definitely the change in both its extension of time and length of river, it suits his method of first collecting facts on which to build his theories. It, of course, makes the problems more complicated, but he has indicated the general methods of meeting them, and that is as much as can expected in a new field such as this is. That where such changes occur, the slope is altered, is evident, but the writer takes these changes of slope as effects; the field in which slope is a primary canse he takes to be quite a different matter.

In that lies the explanation of all the general forms or types shown in the different regimens of different rivers, and the writer does not come to it in the paper presented; he has carried the subject no further than the case of the flume, where the grade was not even considered. He offers, however, a system in which these regimens may be determined, and, indeed, which promises to bring rivers into a much more exact field of calculation than that found in any other line of hydraulics; certainly the more precise determination of m is a subject for further experiment, and the application of the equation $\frac{d\,Q}{d\,h} = m\,W$ must be varied to suit the cases. But taking the given regimen of the Missouri River from Kansas City to St. Charles as an example, perhaps something of the dynamics, in which this, as a whole, stands as an equilibrium, may best illustrate his view of it.

First, the energy of the discharge and fall in this reach varies from about 750 000 H.-P. at low water to about 15 000 000 H.-P. at high water. This, then, tears down on an average about 120 000 000 cu. yds. of bank annually, or an amount which, dumped in year after year, is enough to fill this given prism up solid to some 12 ft. above low water in the twenty years in which the Government has been working on it.

That it has not changed it perceptibly, makes the fact very plain that it has simply been built back again on banks and bars in other places. Mr. Seddon.

Of course, a considerable part of the observed sediment in the water lies simply in this action, and for the reach from Kansas City to St. Charles, as a whole, may be said neither to come into it nor to go out of it. But taking this in about the proportion of the sediment that will settle from the water with the least check to its velocity, and which can hardly have come from distant head waters, leaves the true sediment for the year about 130 000 000 cu. yds. carried in suspension. And certainly the percentage of this which may be dropped in the reach is an additional load on the bar building forces.

But a good part of this matter carried in suspension will not settle in a reservoir after standing 24 hours, while, even as a whole, it is but little more than the erosion on this 333 miles of the river, and, for the total alluvial stretch of the Missouri, is a minor fraction in comparison with it. Indeed, the fact of the almost uniform grade on which this river is set leaves little doubt that it has been long ago leveled up to bring the sediment entering it into practically a through transportation; but, in any event, in all such cases the probable proportions are such, that if no changes from erosion are shown in the 10 or 20-year periods, about 1 000 years is the least time in which material effects may be looked for from the matter carried in suspension.

Of course, all this only shows when the long reach of river is taken into view altogether. At a given location the erosion may be small, and the bar building large, while the sediment carried through in suspension is so great in proportion to both of them that anything seems to be possible from it. It is not until the engineer gets away from actually looking at it, and sees, in his mind, the hundred miles of river as a whole, as he sees the orbit of the planet, that he begins to find himself in the presence of the real dynamics of rivers, and can fairly value the processes given, in which these equilibriums may be measured, and their variations traced from river to river, and from season to season; a matter that at first looks as hopeless to him as chaining the distance to the moon, but which, after all, is very easily measured when we know how.

Vol. XXVI. APRIL, 1900. No. 4.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

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SOCIETY AFFAIRS.

PAPERS AND DISCUSSIONS.

ILLUSTRATIONS.

The prices of publications are as follows: *Proceedings*, $6 per annum; *Transactions*, $10 per annum. Postage will be added when *Proceedings* are sent to foreign countries.

American Society of Civil Engineers.

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Special Committees.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 A.M. to 10 P.M.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - - 533 Columbus.
CABLE ADDRESS, - "Ceas, New York."

Vol. XXVI. APRIL, 1900. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

April 4th, 1900.—The meeting was called to order at 8.45 P. M., the President, John Findley Wallace, in the chair; Charles Warren Hunt, Secretary, and present, also, 81 members and 17 visitors.

A paper by William B. Landreth, M. Am. Soc. C. E., entitled "Recent Stadia Topographic Surveys: Notes Relating to Methods and Cost," was presented by the Secretary, who also read a discussion by Emile Low, M. Am. Soc. C. E. The subject was discussed orally by Messrs. John F. Wallace, Basil Magor, R. S. Buck, Henry Goldmark, A. J. Himes, and R. A. MacGregor.

Ballots were canvassed and the following candidates declared elected:

As Members.

Fred William Bruce, St. Augustine, Fla.
Edward Hanson Connor, Leavenworth, Kans.
Robert Walter Creuzbaur, New York City.
Justus Vinton Dart, Providence, R. I.
Joseph Marshall Graham, Baltimore, Md.
Jere Chamberlain Hutchins, Detroit, Mich.
Walter Alexander Rogers, Chicago, Ill.
George Caleb Stoddard, New York City.

As Associate Members.

Warrick Rigeley Edwards, Baltimore, Md.
Arthur Willard French, Worcester, Mass.
Albion Lorenzo Grandy, Grand Rapids, Mich.
John Philip Hallihan, Alamogordo, N. Mex.
Theodore Horton, Boston, Mass.
Clarence William Hubbell, Detroit, Mich.
George Cecil Kenyon, West Derby, Eng.
Wallace Corliss Lambert, Boston, Mass.
Léon Élie Lion, New Orleans, La.
Frederick Clinton Phillips, Little Falls, N. Y.
Harry Harwood Rousseau, Washington, D. C.
George Simpson, New York City.
Harrison Stidham, New York City.
Thomas William Wilson, Buffalo, N. Y.

Announcement was made that the following candidates were elected by the Board of Direction, April 3d, 1900.

As Juniors.

Ernest Payson Goodrich, Brooklyn, N. Y.
Alfred Liebmann, New York City.
Francis Mason, Albany, N. Y.
George Franklin Perry, Washington, D. C.
James Vincent Rockwell, Boone, Iowa.
William Frederick Steffens, New Haven, Conn.
George Clifton Woollard, St. Marys, Pa.

Adjourned.

April 18th, 1900.—The meeting was called to order at 8.30 P M., Vice-President Rudolph Hering in the chair; T. J. McMinn acting as Secretary; and present, also, 80 members and 17 visitors.

A paper by George W. Rafter, M. Am. Soc. C. E., entitled "On the Flow of Water Over Dams," was presented by the author. Communications on this subject from Messrs. George Y. Wisner, Charles W. Sherman and J. L. Power O'Hanly were presented by Mr. McMinn. The paper was discussed orally by Messrs. G. S. Williams, Emil Kuichling, E. A. Fuertes, Edward P. North and the author.

Mr. M. N. Baker exhibited and described a number of photographs of the Austin dam, recently destroyed by flood.

Announcement was made of the death of the following Members: James Colwell Aldrich, elected Member May 7th, 1873; died April 3d, 1900. Bolton Waller DeCourcy, elected Member November 6th, 1889; died April 1st, 1900.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

April 3d, 1900.—The Board met at 8.30 P. M., President Wallace in the chair; Charles Warren Hunt, Secretary, and present also Messrs. Bensel, Hering, Knap, O'Rourke, Ricketts, Schneider, Seaman, Turner and Whinery.

A report was received from the Committee of Arrangements for the Annual Convention.

A Local Committee of Arrangements, consisting of all persons connected with the Society residing in London, was appointed.

The Secretary was instructed to print in the next number of *Proceedings* a copy of Senate Bill No. 2330, with such explanation in regard to it as will enable members to take any action concerning it which seems to them advisable.

Applications were considered and other routine business transacted.

Seven candidates for Junior were elected.*

Adjourned.

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* See page 94.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 A. M. to 10 P. M.

MEETINGS.

Wednesday, May 2d, 1900, at 8.30 P. M., a regular business meeting will be held. Ballots for membership will be canvassed, and a paper by J. M. Moncrieff, M. Am. Soc. C. E., entitled, "The Practical Column under Central and Eccentric Loads," will be presented for discussion. This paper was printed in the March number of *Proceedings*.

Wednesday, May 16th, 1900.—At the suggestion of Mansfield Merriman, M. Am. Soc. C. E., Chairman of the American Section of the International Association for Testing Materials, this evening has been set apart for the informal discussion of the following Proposed Standard Specifications: Structural Steel for Buildings; Wrought Iron; Structural Steel for Bridges and Ships. Committee No. 1 of this Association has in charge this problem, proposed at Zurich Congress, 1895: "To Establish International Rules and Specifications for Testing and Inspecting Iron and Steel."

The American Committee No. 1 has been engaged actively in this work for the past 18 months, and William R. Webster, M. Am. Soc. C. E., its chairman, will place in the hands of the Secretary copies of the proposed specifications on each of these subjects, for distribution to such of our members as are interested and will discuss them. The Committee is desirous of securing the views of engineers generally, in order that any necessary modifications may be made.

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON THE PROPER MANIPULATION OF TESTS OF CEMENT.

A Progress Report of the Special Committee on the Proper Manipulation of Tests of Cement was presented informally by the chairman of that committee at the Annual Meeting (*Proceedings*, Vol. xxvi, pp. 4 and 43), and has now been forwarded by him for publication. (See page 99).

THIRTY-SECOND ANNUAL CONVENTION.

The following subjects will be presented for informal discussion at the Annual Convention to be held in London during the first week in July, 1900.

Members who are unable to attend are invited to send written communications on any of the subjects for presentation at the meeting.

HEIGHT OF BUILDINGS.

(1) What considerations should limit the height of buildings?

(2) Do recent developments in construction, sanitation, intercommunication and economy of administration, warrant the removal of all restrictions?

RECENT PRACTICE IN RAILS.

The progressive increase in weight; the increase in hardness, particularly in carbon; the sections in most general use; the effect of changes in weight, composition and section.

FILTRATION OF WATER FOR PUBLIC USE.

The several processes now used for the removal of objectionable matter; their comparative sanitary effect, cost and reliability.

IXth INTERNATIONAL CONGRESS OF NAVIGATION.

A project for holding the ninth session of the International Congress of Navigation in the United States has been in contemplation for some time; and, with this object in view, two bills have been introduced before Congress to authorize the President of the United States to extend an invitation to the Congress to hold its next session in this country.

These bills are known as "S. 2330" and "H. R. 6262," and are practically identical. The bill before the House of Representatives has been referred to the Committee on Foreign Affairs. The Senate bill has been read twice and referred to the Committee on Foreign Relations, which committee has reported favorably thereon.

The Senate bill is printed below, to bring it to the attention of members of the Society who may be interested, in order that they may take such action as they may think best.

"56TH CONGRESS, 1ST SESSION. } S. 2330.

"IN THE SENATE OF THE UNITED STATES.

"JANUARY 11, 1900.

"Mr. COLLOM introduced the following bill; which was read twice and referred to the Committee on Foreign Relations.

"MARCH 23, 1900.

"Reported by Mr. CULLOM, with amendments.

"A BILL

"To authorize the President of the United States to invite the International Congress of Navigation to hold its ninth session in Washington, District of Columbia.

"Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That the President of the United States is hereby authorized and directed to extend an invitation to the International Congress of Navigation, at its eighth session, to be held in Paris, France, in July, nineteen hundred, to hold its ninth congress in Washington, District of Columbia, United States of America.

"SEC. 2. That the Secretary of State shall prepare for the use of said International Congress of Navigation suitable halls and rooms in the City of Washington for the meeting of said body.

"SEC. 3. That the sum of thirty thousand dollars, or so much thereof as may be necessary, is hereby appropriated, out of any money in the Treasury not otherwise appropriated, and to be available as required and to be expended under the direction and at the discretion of the President of the United States, to meet the expenses incurred by the Government in connection with the ninth session of the International Congress of Navigation."

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON THE PROPER MANPIULATION OF TESTS OF CEMENT.

The Committee on the Proper Manipulation of Tests of Cement is not able at this time to present a final report, and has not been able to make as much progress as it had hoped, owing to numerous circumstances.

The first thing that the Committee did was to prepare and send out a circular asking the opinion of members of the Society, and others who were supposed to be interested in the subject, with reference to the questions involved. Replies to this circular have been received from 45 persons, who may be classified as follows:

Members of the Society	26
Associate Members of the Society	9
Associate	1
Other persons, not members of the Society	9

The number of these replies did not indicate, in the opinion of the Committee, a very general interest in the subject, and the replies themselves differed so very greatly in their recommendations that it appears a difficult matter to formulate a set of recommendations which will meet with general approval. The Committee presents at this time a summary of these replies, believing that it will be of interest to members of the Society. In presenting this report, the Committee, of course, assumes no responsibility, and does not express the opinion of its members, but simply presents a summary of the replies to the circular.

In considering the questions in the circular and the replies thereto, it must be remembered that the present Committee was not appointed to formulate tests, but simply to formulate the method of manipulation of the different tests. Some of the questions in the circular, however, were made broader than the work assigned to the Committee would appear to warrant, because it was desired to ascertain as broadly as possible the opinion of the members of the Society on the questions involved. In making its final report, the Committee will, of course, limit itself strictly to its proper functions. If the Committee is continued by the Society, it hopes that it may be able to find time during the coming year to perform some experiments of its own, and at the next meeting to present a definite report of its own.

For the Committee,

G. F. Swain,

New York, January 17th, 1900. *Chairman.*

Question 1.—In the works which you have carried out, how much cement have you been willing to accept on the results obtained with a single sample?

The majority of those replying to this question favored testing

about one sample from every ten barrels—thirteen persons specifying this number. One was in favor of testing one sample in six barrels for tension and making a chemical test of one in 100 to 300 barrels. One tests every barrel, if doubtful, otherwise every third or fourth barrel. One says one sample in ten for well-known brands, and one in three to five for unknown brands. Two say one for every car load for known brands. One says one in five barrels at first, and if the product runs uniformly, one barrel in ten. The general opinion is that the reply to the question would depend upon the kind and reputation of the cement, although three replies stated definitely that it does not so depend.

Question 2.—What method do you recommend for obtaining a sample from a package?

There is a general agreement with reference to this question, the general practice being to use an auger or sugar trier and to take a sample from the center of the package. Several members, however, are in the habit of taking the sample from the outside or from different parts of the package in order to get the average quality.

Question 3.—Do you mix cement taken from several packages to obtain a sample to use in testing, or are the samples from the several packages kept distinct?

There is a general unanimity to the replies to this question. The great majority of those answering being in favor of keeping the samples distinct, only five being in favor of mixing them. One recommends keeping them distinct for tensile tests and mixing them for chemical tests and tests of soundness; while two recommend just the reverse. One recommends keeping them distinct for tests in the field, but mixed when making an elaborate series of tests as a matter of investigation. One recommends keeping them distinct for tests of fineness, time of set, and checking, and if these results are uniform, mixing the samples for tensile tests with sand, but not for neat tests. One simply recommends keeping samples distinct for neat tests and mixing them for mortar tests.

Question 4.—When do you consider a chemical analysis essential or desirable?

As a result of the replies to this question, the average opinion seems to be that a chemical analysis is desirable in the first examination of any cement and for purposes of control if any of the physical tests are unsatisfactory; that it is also, of course, necessary for the manufacturer in order to control the uniformity of his product; that it is not, however, by any means essential for the engineer in the field, except as a general indication, and except in cases where the cement is to be used in sea water, in which case the presence of certain elements may be injurious.

Professor Henry Carmichael writes as follows to H. A. Carson, M. Am. Soc. C. E: "Chemical tests are of service: (*a*) in establishing the composition and identity of sample which has been selected as the standard to be used in any particular work, and in detecting any departure from the standard sample which is specified; (*b*) in demonstrating the presence of hurtful ingredients such as magnesia, free lime in excessive amount and sulphate of lime; (*c*) in demonstrating the presence of mere adulterants such as fine sand, ashes, pulverized furnace slag, etc; (*d*) in ascertaining the amount of carbonic acid and moisture present, which indicate to what extent the cement has been 'air slacked,' and, under circumstances, whether the materials have been properly ignited in the manufacture of the cement. It is to be

noted that the chemical composition alone may show the cement to be entirely unsuited for a particular use, as for instance, that it is Rosendale or a mixture when a pure Portland is required, and again it may show that the composition is consistent with that of a good cement; but it does not prove it to be good cement unless supplemented by mechanical tests, since the temperature at which the cement is formed and the physical state of the cement affect the quality of the product equally as well as the chemical composition."

Professor J. A. L. Waddell writes as follows: "I consider a chemical analysis of cement essential or desirable only when trouble is being experienced with that particular kind of cement. Such a test is more useful to the manufacturer in securing a uniform product than it is to the engineer in detecting any undesirable qualities in the cement. Good cements differ so widely in composition that no standard based upon chemical analyses can be established. Neither can such analyses be used as a test for soundness, because they do not distinguish between lime and magnesia when so combined as to give added strength, and the same elements when uncombined, in which state they prove a source of danger."

One member considers that after careful investigation of a cement, all physical tests might be dispensed with, and reliance based entirely on the degree of calcination, fineness and the chemical composition, but most engineers would hardly agree with this opinion. Classifying the replies received, three reply "for all works of importance"; two reply "always" or "indispensable"; one, "wherever possible"; one, "every shipment"; five, "in general, unnecessary"; one, "only where adulteration is suspected"; three, "only where trouble is experienced"; four, "only for new cements"; four, "only for the manufacturers"; four, "in salt water."

Question 5.—What elements or compounds should be determined?

A number of members unite in considering magnesia, sulphates and adulterations the only substances necessary to determine, while others add lime, silica, alumina and iron. making practically a complete analysis. In certain instances, no doubt, it may only be necessary to determine the lime, magnesia and sulphates, while in others a complete analysis may be desirable.

Professor Henry Carmichael says: "Hydraulic cement consists of a double silicate of lime and alumina (including iron oxide), which is readily soluble in dilute hydrochloric acid, leaving little or no insoluble residue. In addition to the soluble silica and the oxides of calcium, aluminum and iron, good cement contains traces of the oxides of magnesium. sodium and potassium, together with traces of carbonates, sulphates, chlorides and combined water, and finally minute amounts of insoluble sand or cinder.

"A larger amount of magnesium oxide than 2% is regarded with suspicion. More than a trace of sulphate of lime is objectionable under ordinary circumstances, since, while it facilitates the setting of the cement, it prevents it reaching the ultimate strength of a good Portland. Free lime is supposed to cause the cracking of concrete after it has set. Any considerable amount of sodium of oxide indicates the presence of silicate of soda, which is intentionally added for hastening the time of setting, and in some of the applications of cement is a desirable ingredient."

Question 6.—What do you consider the best methods of determining these compounds with sufficient accuracy?

The general consensus of opinion of those best qualified to judge

appears to be that the chemical analysis must be done with great accuracy in order to be of any value, as very slight variations in the composition of a Portland cement may have a very decided effect upon its quality. It is not necessary here to enter into the methods of analysis in detail, although some of the replies do so. For comparison, however, we quote two of these replies:

Professor Henry Carmichael says: "The sample is ground fine in an agate mortar. One gram is carefully weighed out in a shallow porcelain dish and well covered with a 3% solution of hydrochloric acid. After several hours the cement should completely dissolve in this acid with the exception of a small amount of sand, mostly black cinder from the fuel employed in making the cement. The residue, if any, is filtered off and determined. The clear solution is evaporated to dryness on a water bath in a flat dish. Hydrochloric acid is poured over the dry residue, and acid is then evaporated. Add a few drops of same acid, and again drive off acid. Moisten residue again with same acid and boil up with pure water. The silica is rendered insoluble by the above operation and can be filtered off and weighed. The silica which thus dissolves in dilute acid, and is in turn rendered insoluble, is the silica which is available in the setting of the cement. The filtrate from silica is boiled with a few drops of nitric acid, and pure ammonia is then added which precipitates the oxides of iron and aluminum. With the ammonia is added also ammonium chloride in sufficient quantity to retain the lime in solution. After boiling for some time, the oxides of iron and aluminum are filtered off and after drying are ignited and weighed. After weighing, the two are dissolved by ignition with potassium bisulphate and after the solution and reduction of the iron sulphate by metallic zinc, the amount of iron oxide is ascertained by titration with potassium permanganate. The oxide of aluminum is calculated by difference. To the filtrate from the iron and aluminum oxides is added in slight excess ammonium oxalate whereby the lime is precipitated as calcium oxalate which is filtered off, ignited at a dull red heat in platinum crucible and weighed as carbonate.

"The filtrate from oxalate is evaporated to dryness, covered with glass and strong nitric acid gradually added until the ammonium salts are destroyed. The solution containing magnesium and the alkalies is then evaporated to dryness in porcelain crucible, moistened with water, a few drops of mercurous nitrate added, water evaporated, covered and ignited at low redness. The magnesium oxide, which is now rendered insoluble, is filtered off, and, after ignition, weighed. The filtrate, after the addition of a few drops of strong hydrochloric acid, is evaporated to dryness and weighed as combined chlorides of the alkalies. The chlorides are weighed and then dissolved in small amount of water. Platinum chloride is added to solution and some alcohol and ether is added to render the potassium double salt less soluble. The latter is after some hours filtered off and weighed. The potassium computed as chloride subtracted from combined chlorides gives, in the absence of sulphates, by difference the amount of sodium chloride.

"Having thus as above determined the principal bases present and silica, other portions of the sample are taken for the determination in the customary manner of the sulphates and chlorides.

"Another weighed portion ignited in platinum at a low red heat gives by loss the amount of combined water present, and if after the weighing of the crucible it be again ignited at an intense white heat, the further loss is that of the combined carbonic acid.

"By an amplification of the above method all other acids and bases are readily determined.

"It is, of course, not necessary that the above *quantitative* course shall be pursued in all cases, and it will oftentimes be sufficient if a few qualitative tests alone are employed, which may be applied according to the particular issue involved.

"The question of free lime present is difficult to determine, and it is found to be a useful indication to titrate the lime present with a deci-normal hydrochloric acid solution with phenol-phatalein as indicator. It will be noted that the lime combined as silicate is slowly neutralized, while the free lime is at once neutralized by the dilute acid. Quantitatively it is best determined by comparison with the amount normally present in standard makes of cement."

R. L. Humphrey, Assoc. M. Am. Soc. C. E., writes as follows: "The following scheme for analysis has been adopted by the writer. One-half gram of the finely pulverized sample, dried at 100° C., is thoroughly mixed with four or five times its weight of sodium carbonate, and fused in a platinum crucible until CO_2 no longer escapes; the crucible and its contents is placed in a beaker and twenty or thirty times its quantity of water, and about 10 c. c. of dilute HCl is added; when complete solution is effected, it is transferred to a casserole and placed on a water bath, and evaporated to dryness several times. The mass is taken up with dilute HCl and water, heated for a short time and filtered, washing the residue on the filter thoroughly with hot water. The filter is dried, ignited and weighed. This weight (less ash) gives the amount SiO_2.

"The filtrate is brought to boiling, and ammonia hydrate is added in slight excess, the boiling is continued until the odor of ammonia is no longer perceptible. Filter and wash. Re-dissolve in hot dilute HCl, again precipitate with ammonia and filter through the previous filter and wash with boiling water. The precipitate dried, ignited and weighed, less ash, gives the amount of Al_2O_3 and Fe_2O_3.

"The iron is determined volumetrically by fusing the ignited precipitates of alumina and iron with de-hydrated potassium sulphate in the platinum crucible; it is then dissolved in sulphuric acid and titrated with potassium permanganate.

"The filtrate from the iron and alumina is heated to boiling, and boiling ammonium oxalate is added until a precipitate is no longer formed. After boiling for a few minutes, it is set aside for a short time; when the precipitate has settled perfectly, decant the clear liquid through a filter, wash by decantation, dissolve the precipitate in hot dilute HCl, using as small a quantity as possible to effect a complete solution, heat to boiling and add ammonia, heat on a water bath for a few minutes; when the solution clears filter through the previous filter, wash thoroughly with hot water. Dry the precipitate, ignite to constant weight, and weigh as CaO; or determine the lime volumetrically by titration with potassium permanganate.

"The thoroughly washed precipitate of calcium oxalate is dissolved in hot dilute sulphuric acid and the solution is titrated with potassium permanganate.

"The filtrate from the calcium oxalate is made alkaline with ammonia and 30 c.c. of solution of hydro-di-sodium phosphate is added; the whole is set aside in a cool place for 24 hours; it is then filtered and washed about fifteen times with ammonia water solution (1 : 5). Dry the precipitate on the filter, brush on to a large watch glass, burn filter on the lid of the weighed crucible. When the carbon is

consumed, transfer the precipitate to the crucible and ignite to dull redness, keeping the crucible covered. If the precipitate is not perfectly white on cooling, moisten with a few drops of nitric acid, evaporate and ignite to dryness; weigh as magnesium pyrophosphate and calculate to MgO.

"Sulphuric acid. This is determined in a separate portion. Weigh out about five grams and treat as in the regular analysis, separating the silica; the filtrate is heated to boiling and boiling barium chloride is added; the boiling is continued for ten minutes; when the precipitate has subsided, filter. The precipitate is thoroughly washed in hot water, dried, ignited and weighed as barium sulphate and calculated to SO_3.

"Carbonic acid. This can be determined with sufficient accuracy by means of the ordinary extraction apparatus."

Questions 7 and 8.—7. Are microscopical tests of value, and, if so, when? 8. What power microscope is required, what observations should be made, and what are the indications?

The general opinion in reply to Question 7 is that a microscopic analysis is in general unnecessary, six replies being to this effect. One writer thinks it is desirable for the determination of adulterants; one thinks it is sometimes necessary, and one that it is only necessary in order to determine the cause of a failure of cement by observing the character of the crystals. We quote several replies.

Mr. C. A. Richardson writes: "I consider microscopical tests to be of value in determining the nature of the coarse material which is left on sifting cements, and in determining whether the sulphate of lime which is often added to Portland cements, is thoroughly mixed. A microscope having a 1-in. and ½-in. objective, with suitable illumination, such as the "Star Model" Beck, is all that is necessary. If the sulphate of lime is too coarse or in balls, it will be revealed by the appearance of white spots in the cement. If the coarse material, obtained on sifting, is of a different nature from that which merely well burned but unground clinker will show, it may reveal the presence of overburned clinker or admixture of some material foreign to a normal Portland cement."

Professor Henry Carmichael writes: "Very little use has as yet been made of the microscope in the practical testing of cements although it is thought that more use could be made particularly in the examination of the hardened cement by methods now well understood in the examination of the crypto-crystalline rocks. The hardened cement must be cut for its examination into sections as thin as writing paper or thin enough to be transparent. Such sections are made by first grinding and polishing one side of a thin fragment, which is then on this side cemented by Canada balsam to a short strip of glass. The rough side is then ground down and when thin enough polished, after which a very thin glass cover is cemented over it with the same medium. The mass will be found to consist of round granules and crystals. The free lime appears in the form of hexagonal crystals. During the hardening of the cement the silica gradually unites with the calcium and the aluminum oxides to form in part delicate needles to which the toughness of the concrete is undoubtedly in part due. The difficulty in preparing sections of the requisite thinness is one of the principal obstacles to the examination of cements by this method. The changes which take place upon addition of water may also less satisfactorily be observed if the finely ground cement is mixed with water under a glass cover with cemented edges to prevent evapora-

tion. The growth of the hydrous silicates upon which the setting of the cement depends can thus be directly observed. If it be desired to examine the dry cement under a microscope it should be mounted with oil of cloves under thin glass cover to render the particles more transparent. So far as the writer is aware, the microscope is distinctly inferior to the chemical tests in determining the relative value of cements.

"In such investigations a microscope will be found useful which has lenses magnifying about 150, 250 and 600 diameters. One of 80 diameters is also convenient."

Mr. R. L. Humphrey writes: "Microscopical tests are not of sufficient importance to be made a part of a regular system of tests. For original research or scientific investigations in well-equipped laboratories, they may furnish considerable information. In the examination of thin sections of hardened cement, a microscope with high powers is required. For examination of the dry powder, a hand magnifying glass or a microscope of low power should be used. The dry powder for such an examination should be freed of the fine dust by passing through a No. 200 sieve. The examination of the residue on this sieve often reveals the presence of foreign matter (adulteration), or the character of the grains indicates under or over burning. The suspicious grains can be isolated and further examined under the microscope, making a chemical analysis of the grains thus separated."

Question 9.—What sizes of mesh should be used in testing fineness of Portland cement? What for natural cement?

The answers to this question are very variable and can hardly be summarized. We quote, however, a few of the replies received. Professor J. B. Johnson says: "I am thoroughly satisfied that no fineness test by sieves is adequate. If we can do no better, then use the *finest practicable*, which is, I think, 120 meshes per linear inch. To use 150 mesh, or finer, takes too long. *There is no significance to be attached to results from coarser sieves.* This the Committee should emphasize. Possibly 150-mesh could be used. I think the Committee should try to find a practicable washing method of testing fineness. *This matter of fineness is the most important question for the Committee to decide*, in my opinion. It is this alone which determines the money value of a given kind of cement."

Professor Clifford Richardson says: "I think that all cements should be sifted upon the sieves ordinarily known as the 50, 100 and 200-mesh. The brass-wire cloth, which is in use in these sieves, should be, for the 50-mesh, of wire 0.21 mm. in diameter; for the 100-mesh, of wire 0.12 mm. in diameter, and for the 200-mesh, of wire 0.06 mm. in diameter. These brass-cloths are known in the trade as, for the 50-mesh, No. 35-6 E wire; for the 100-mesh, No. 39-6 E wire, and for the 200-mesh, extra fine brass-wire cloth, 0.0025 in. diameter wire. There is no great difficulty in obtaining a number of 50-mesh sieves which will agree in the amount of material which they will pass from a given sample, but with the 100 and 200-mesh sieves there may be a great variation between sieves made from the same roll of cloth. This is due, primarily, to the fact that, in stretching the finer cloth upon the frame, it is very apt to be pulled out of shape and the size of some meshes increased and others diminished, and the requisite care is not taken by the ordinary manufacturer to rectify these defects. I have at last been able to obtain, from Howard & Morse, 1197–1211 DeKalb Avenue, Brooklyn, N. Y., the finer sieves, which they have made with such care as to be quite satisfactory. There are also other defects due

to the unavoidable defects in the weaving of fine cloth. These can only be got rid of by careful selection of the piece to be used in the screen. The wire cloth should be plain woven and not twilled."

A great many sieves appear to be in use, varying from 50 to 200 meshes per linear inch. The size of wire is apparently not generally specified or known, but standard sieves of certain normal mesh are used. Several members refer to the tediousness of using very fine sieves, and while they recognize the fact that it is only the material that is an impalpable powder which has any cementing value, they consider that just as good results in practice may be obtained by using meshes not finer than 100 for Portland and 50 for natural cements.

Question 10.—What should be the diameters or sizes of the screens?

From 6 to 8 ins. in diameter seems to be the size of screen preferred by most of the members, although some specify as small as 4 ins. and others as large as 15 ins. A number call attention to the use of nests of screens which allow the sifting through all of them to be done at one operation.

Question 11.—How large a sample should be tested?

The replies to this question are quite variable, ranging from 1 oz. to 1 lb. A greater number of the replies indicate the use of 100 grams, or about 4 oz., than any other quantity.

The proper quantity has some relation to the fineness of the sieve used, as it is, of course, tedious to attempt to sift a large quantity through a very fine sieve.

Question 12.—Should any machine for shaking be used, and if so, what form, and what should be its manipulation?

A considerable majority of those replying to this question consider that the use of a machine is not necessary, although a few of these consider it desirable. Seven writers definitely advise a machine, two reply that no machine that they know of is satisfactory, while three, although they know of no machine, think a machine could be made satisfactory.

Captain F. V. Abbot makes this suggestion: "I suggest having the sieve rest on a steel ring with a saw-tooth bottom. This ring should have guides at two sides which allow it to move vertically but not to revolve. This steel ring should rest on another with steel pins which enter the saw-tooth edge of the upper ring. The pin ring should revolve about its vertical axis slowly."

Mr. R. L. Humphrey replies as follows: "The work of shaking the sample through the sieve is best done by hand, but as this is a very tedious operation, especially where the number of samples to be sieved daily is large, the writer designed a mechanical sifter which is in use in this laboratory. It is operated by a small electric motor. The frame, holding four sets of sieves, is given a tilting motion to and fro. The driving pulley makes 100 revolutions per minute, and the frame 200 movements to and fro. The crank disc has a throw of 1½ ins., and the rapid motion imparts a jerking movement to the sifting frame. Four samples are passed successively at one operation through the No. 100 and No. 200 sieves; these sieves fit into each other, and have a top cover and bottom pan, the samples being placed on the No. 100 sieve and passing through this sieve and the No. 200 sieve below it, are caught in the pan."

There appears from the replies to be no reason why hand shaking is not just as good as machine shaking. A machine would, of course, be desirable where a large amount of testing is being done, simply as a labor-saving device.

Question 13.—How long should the shaking be continued?

The answers to this question are necessarily somewhat indefinite, a great majority simply stating that the shaking should continue till no more cement goes through. The length of time depends, of course, upon the fineness of the sieve and the character of the cement.

Captain F. V. Abbot replies as follows: "The saw ring should be 13 ins. in diameter, and should have 100 teeth. The pin ring should revolve ten time a minute for ten minutes. This gives the sieve 10 000 jars. The saw teeth should be about ¼ in. deep, so that each jar should mean a vertical drop of the sieve of about that height."

Mr. R. L. Humphrey states that the shaking, either by hand or machinery, should be continued until not more than one-tenth of 1% passes through the sieves after five minutes' continuous shaking.

Question 14.—Should there be any difference in manipulation for fine and coarse screens, or for different kinds of cement?

The general reply to this question is that there is no difference. Of course, there is a difference in the time required for screening. One member suggests using, with fine screens, successive small quantities of cement.

Professor Clifford Richardson makes the following remark: "The manipulation of fine screens requires much more care than for the 50-mesh, but I do not know that there is any difference between different kinds of cements. With the fine screen, it is necessary that it should be jarred against a table, or something firm, frequently during the process of sifting, and that a stiff bristle varnish brush should be used from time to time to break up any lumps and clear the cloth when it becomes caked."

Mr. R. L. Humphrey replies as follows: "For any sieve having less than 100 meshes per inch, the sample can be sieved without special preparation. For sieves having a greater number of meshes the sample should be thoroughly dried, at a temperature of about 130° C., before sieving. Otherwise the cement often clogs the meshes, thereby rendering the sieving very slow and difficult."

Question 15.—Apparent density or weight per cubic foot. What is your opinion of the value of this test?

Out of 28 members replying to this question, 19 consider this test either valueless or of no importance. Professor Newberry considers it useful for distinguishing pure Portland cement from mixtures, and one or two others make the same remark. Several replies add that the requirement of a certain weight per cubic foot is a direct encouragement to coarse grinding. Three members consider the test "valuable if properly made"; two consider it of some value in connection with the color, fineness or specific gravity.

The general result seems to be, in the opinion of the members, that the test is of little practical value, and while in connection with other elements it may give some indications, yet the variety of conditions is so great and the liability of divergence in the results so large that it is practically scarcely worth while to make it. Professor Clifford Richardson tersely states: "I think that this test is of no value as it is dependent upon too many varying conditions."

Question 16.—What apparatus do you prefer, and how should it be used?

Only a few members reply to this question, since there are but few who consider the test of any value. S. B. Russell, M. Am. Soc. C. E., replies as follows: "Am now trying a new method, which I hope will

prove good. A briquette is made of neat cement and water, just as for a tensile test. After setting in air for 24 hours and in water for 6 days, or until the period of rapid chemical change is past, the briquette is weighed in air and in water, to determine its volume. It is then dried thoroughly at 212° Fahr., to get its dry weight. From the volume and dry weight we get the specific gravity. A piece of the briquette is now weighed and then heated in a platinum crucible until thoroughly calcined to a constant weight. A sample of the ungauged neat cement is also weighed and calcined to a constant weight. The difference between the percentage loss of the briquette and the ungauged cement shows how much water was taken up in setting. Correcting the specific gravity of the briquette by the amount due to the water taken up in the setting, we get the weight of the original cement in the briquette. From this we can compute what volume 100 lbs. of cement would have if made into briquettes. This, in my opinion, is what we want to find out. To compare cements, we must know which will give the greatest volume of compact mortar for a given weight. The operation of this test is somewhat difficult to describe, but is simple in the doing."

One or two others describe the well-known hopper apparatus. It is well known that considerable care is necessary in order to obtain uniform results with this or any other apparatus.

Question 17.—True density or specific gravity. What is your opinion of the value of this test?

Of 23 writers replying to this question, 10 think this test has very little value. One thinks it should be used with great caution; one thinks it is of considerable value in examining a new brand of cement; one considers it valuable in connection with weight per cubic foot, and one advises it wherever chemical tests are made for the purpose of showing the degree of calcination, and six consider it valuable. M. J. Butler, M. Am. Soc. C. E., considers it the only test by which we can determine the proper calcination of the clinker, while T. C. Hatton, M. Am. Soc. C. E., finds the results so far at variance with each other that no standard could well be established.

Mr. J. L. Allison says: "In my opinion this test is of much value, in that its indications show whether or not the materials have been sufficiently burned in the kiln. A low specific gravity would show either underburning or adulteration."

Professor Clifford Richardson replies as follows: "The value of this test is considerable in examining new brands of cement. I have, however, never met a high-grade Portland cement that would not show satisfactory specific gravity. With natural cements very different densities will be found for the different brands. All things being equal the cement of the highest specific gravity is the most desirable. I have shown, in some recent papers of mine in *The Brickbuilder*, that the densest natural cement in this country is the Rosendale."

Undoubtedly, the determination of the specific gravity should be made in all well-equipped laboratories, but the replies indicate that its results will not be of much practical value and the tests will generally be out of the reach of ordinary practice.

Question 18.—What apparatus do you prefer, and what is the method of manipulation?

Several members describe in more or less detail the apparatus used for this test. Mr. M. J. Butler describes an ordinary specific gravity bottle which he is in the habit of using, weighing it first full of water and again when filled with water to which is added 100 grains of

cement. Mr. J. L. Allison describes a volumometer which is used with turpentine. Mr. D. Molitor describes Schumann's apparatus, also using turpentine, and Mr. R. L. Humphrey describes Le Chatelier's apparatus which is used with benzine. Professor C. Richardson says: "I always obtain the density of a cement in a picnometer on a chemical balance with freshly distilled kerosene of known specific gravity. The ordinary method of determining the specific gravity of cements by displacement I have found very unsatisfactory on account of the difficulty of regulating temperatures and excluding air bubbles."

Question 19.—What kind of sand should be used in tests of mortars? Would you recommend a natural sand or crushed quartz?

Of 34 writers replying to this question, 20 prefer crushed quartz and 12 natural sand, while two do not use sand tests. We quote several of the replies to this question:

Professor Clifford Richardson says: "The kind of sand to be used in making tests with cements will depend upon whether an absolute and standard test is to be made of the cement, or merely relative tests with others made in the same way in the same laboratory, or whether the tests are to be made rather of the mortar to be used than of the cement itself alone. In the former case, I should recommend crushed quartz of a size to be agreed upon, and am not in favor of the natural sand such as is used in Germany. For ordinary comparative tests, however, a natural sand is entirely satisfactory where a uniform supply is to be had, if it is sifted so as to exclude the coarser and finer particles. In many cases, I have found it of value to test the cement with the sand to be used upon the particular work in which the cement is to be employed."

Captain F. V. Abbot says: "Crushed quartz should be adhered to. This has been the standard so long that a change would make past experiments which are now in print and available to the whole profession valueless in comparison with future results. I have personally used screenings from crushed granite, as this was always conveniently available."

J. P. Snow, M. Am. Soc. C. E., says: "We have never made a practice of making tests with sand. The strength is so small that the accidents of testing are likely to cause great eccentricity in the individual tests and the time needed to get proper results precludes its use in practical construction work. Complete laboratory tests for a scientific purpose demand tests with sand, but the regularity of manufacture of any given brand can be ascertained by testing neat samples."

Mr. R. L. Humphrey says: "It is preferable to have a crushed quartz of assorted sizes. Owing to the difficulty in obtaining such a sand, it is best to adopt as standard a sand composed of grains which pass a sieve having 400 meshes per square inch and which are retained on a sieve having 900 meshes per square inch. Crushed quartz makes an admirable sand for this purpose, and gives the best results; but as it is expensive and is not always available, the writer believes it would be better to sieve the local river or bank sand to a standard size. The present American Society Standard sand is satisfactory."

Several of the members are in the habit of using for tests the same sand that is to be used in the work, but such a practice is, of course, not possible for standard tests. If a standard uniform sand were easily accessible to all engineers, it would probably be preferred. This condition of things exists in Germany, where a standard sand is

prescribed. The impossibility of fulfilling this condition in this country is probably the reason why the majority of members prefer to adhere to the crushed quartz previously recommended by the Society.

Question 20.—What fineness should be specified, and what degree of variation in size of grain should be allowed?

Of 27 writers replying to this question, 14 prefer to adhere to the present standard of the Society—that is, that the sand shall pass a No. 20 sieve and be caught on a No. 30. The others give varying replies. One considers that there should be finer particles in the sand, and recommends that about 70% consist of the standard sand while the remaining 30% will pass the No. 20 and will be caught on the No. 50.

Question 21.—Should the same method of preparation be used for each test?

Twenty-four replies advocate the same method of preparation for each test; three reply simply "no"; two, that, in tests for soundness, there should be a greater proportion of water than in tensile tests; one, that in tests for setting and soundness, there should be less water than for tensile tests; and one replies that it is an open question. We quote one of the replies, that of Professor Clifford Richardson:

"It is certainly an open question whether the method of preparation of the paste or mortar for determining the time of setting should be the same as that for the tests for soundness and strength. I have been in the habit of preparing them in the same way, but as the German rules provide an essentially different paste for determining the setting from that of the mortar for making test pieces for strength, there is excellent authority for so doing. The time of setting is so dependent upon the consistency of the mortar that it seems to me it is easier to prepare dry mortar uniformly than moist paste, and consequently I prefer this method."

With reference to this, we note that the principal difference in the German method is that for tests of set and soundness, neat cement is used, and for tests of strength, a mortar of cement and sand.

Question 22.—How should proportions be stated?

Eighteen replies are that proportions should be stated by weight, five that they should be stated by volume. Three advocate stating the proportion of sand to cement by volume, and the percentage of water by weight. One advocates stating proportions by weight for Portland cement and by volume for natural; and one writes that the proportion should be stated by volume, but mixtures made by weight.

Question 23.—What should be the consistency of the pastes and mortars for the various tests, and how may this consistency be specified and determined in order that similar results may be obtained by all operators?

The replies to this question, which is a very important one, are quite diverse, and it is desirable to quote from a number of them. It may be stated, however, that 11 replies indicate a preference for the use of practically as small an amount of water as possible in the preparation of the paste, while four express a preference for the German Standard, one of these replies, however, coming from a German manufacturer.

Mr. T. C. Hatton says: "I have during the past three years established the following standard for consistency, no matter what cement is to be tested, mixing sufficient material only to form one briquette at a time. For Portland cement. Quantities being taken in ounces, the proportions of cement, sand and water are as follows: 10, 0, 1¼; 5, 5,

$1\frac{3}{16}$; $3\frac{1}{2}$, 7, $\frac{3}{4}$; $2\frac{1}{2}$, $7\frac{1}{2}$, $\frac{3}{4}$; $1\frac{1}{2}$, $7\frac{1}{2}$, $\frac{1}{2}$. For natural cements: 8, 0, $1\frac{3}{8}$; 4, 4, $1\frac{5}{16}$; 3, 6, $\frac{7}{8}$; $2\frac{1}{4}$, $6\frac{3}{4}$, $\frac{3}{4}$. I arrived at this result after a vast deal of trouble in trying to have my assistants exercise their judgment for securing proper consistency without securing reliable results. Since establishing this standard, I have had no cause to change it."

Professor Clifford Richardson, whose reply is typical of several others, replies as follows: "In regard to proportions of water for the making of briquettes for tests for soundness and strength, I should say that as little should be used as experience shows will bring the resulting mortar into a plastic condition when rammed into the mould and allow a certain moist appearance upon the surface of the briquettes. The ordinary suggestion that the mortar so prepared shall have the usual appearance of moist loam seems to me to describe the condition better than any other. There are, however, certain conditions connected with the making of dry mortar from the different cements which can only be learned, not described satisfactorily. Some cements require a much longer working with a small amount of water, than others, and at first seem to be much too dry when the proper amount is used, but with continued kneading with a spatula a point will be reached when the paste or mortar seems to ripen and be ready for the mould. I should say that experience alone would be the only means which will teach a novice how to make a proper mortar. I know of no device for determining when the proper consistency has been obtained. This point must be decided by the eye and by the feeling of the mortar under the spatula or trowel. The manipulation of any cement into a sand mortar must be judged very largely from the way it acts in the neat mortar. and a quick-setting cement will, of course, require different manipulation from one that is slow-setting."

Captain F. V. Abbot says: "The only fair test of a cement is the greatest strength it can be made to attain. Experience shows that this requires a different consistency for almost all brands. The right proportion for each brand should be ascertained by careful experiment at the start, and then this proportion should always be used for this brand in subsequent tests."

R. W. Lesley, Assoc. Am. Soc. C. E., says: "In my mind, no absolute specification can be made by which the consistency may absolutely be specified and determined, and by which similar results may be obtained by all operators. As long as the personal equation exists, either in the manipulation of the mechanical mixing apparatus, or in the mixing of briquettes by hand, variations will constantly occur, due to this personal equation."

C. S. Gowen, M. Am Soc. C. E., says: "In a general way, enough water should be used to make the mass plastic after a mixing so thorough that no air bubbles will show. The required plasticity may be defined as one resulting in a mass that will stand alone when shaped with a trowel and yet will spread slightly when the mixing slab is rapped with the trowel."

J. P. Snow, M. Am. Soc. C. E., replies: "If mixed quite dry, an apparent effervescence occurs when the briquette is placed in the water. Sometimes this is so active that the samples partially crumble and are lost. If mixed quite wet, the setting is retarded and 24 hours is hardly long enough to get regular results. Again, the strength is so small that the accidents of manipulation have too great a value in the results, as noted in referring to sand tests. We aim to secure a consistency that will cause no dangerous effervescence and that will admit of tamping into the mould without much jelly-like quaking. It is a very

stiff paste; much too dry to use comfortably as mortar. For mixing pats for tests of soundness and time of setting, the paste is made softer than the above, or about as stiff as common putty. We depend wholly on the operator's judgment to fix upon the correct consistency. Slight variations are of but little account in tests for soundness, but they must be carefully guarded against in making tests for tensile strength. These variations are quickly perceived when ramming the cement into the mould. A few trials will enable the operator to fix upon the right amount of water for a strange brand of cement and the slight variations that will unavoidably occur when working with a known brand had best be noted as 'damp,' 'dry,' etc., along with their number at the time the briquettes are made."

Mr. R. L. Humphrey replies: "The writer believes that the proper consistency for neat cement should be such that the paste when moulded into a ball and allowed to fall from a height of about 1 ft., should not flatten perceptibly or crack. Such a consistency depends upon the length of time the paste is worked. The greatest uniformity is obtained by working slow-setting cements about five minutes and quick-setting cements about two minutes. The writer has used, with excellent results, the normal-consistency apparatus illustrated in the accompanying pamphlet, for the determination of the percentage of water to be used " (Vicat needle apparatus).

All the foregoing replies indicate the preference for a stiff paste. Several members, however, disagree with this suggestion. Thus, Mr. J. L. Allison replies as follows: "In my opinion, the greatest uniformity in results could be secured by using pastes of such a consistency that the moulds could be filled without using any compression. I have found that satisfactory results could be obtained by having the paste plastic enough to flow to all parts of the mould under the action of the point of a trowel moved edgewise (the trowel held vertically). The consistency of such a paste may be measured by its resistance to the penetration of a solid cylinder; and the consistency to which a cement is to be mixed for testing may be specified by requiring it to be mixed with such a proportion of water as to give a paste into which a solid cylinder, of prescribed weight and section, shall penetrate a specified depth, the paste to be contained in a mould of greater depth than the specified penetration. I have used a cylinder 10 mm. in diameter, weighted to 300 grams, used on paste filling a mould of 80 mm. in diameter and 40 mm. deep. A penetration of 34 mm. was the standard. This was determined in the first place by mixing pastes to a satisfactory consistency and noting the penetration. I have not tried any other sizes or weights of cylinder or depth of mould."

Professor C. B. Smith writes as follows: "Use 20% water with 3 to 1, obtaining soft mortars easily moulded and giving low tests. The results are uniform, easily duplicated, and accord with condition of mason's mortar. I believe the French methods lean to this idea. It is standard for the Canadian Society of Civil Engineers, and I, myself, believe in it fully."

Mr. A. G. Fogg writes as follows: "My practice has been to add just enough water to make the mixture plastic when well mixed up and worked. Then I ram it into the mould. I use about 18% of water in neat Portlands and 26% in neat Rosendales. I am not in favor of this degree of consistency. The reason why I adopted it was because I wanted the cements used on my work tested as high as any in the market, and ramming is practiced by a great many testers. The

less water that is used and the harder the paste is rammed into the moulds, the higher the test. This practice is a bad one, because it renders worthless the tables which are printed giving comparative tests of different cements. Each cement dealer has his own comparative list.

"Since we are going to have a uniform standard, I see no advantage in ramming at all. I think it is a source of trouble and disadvantage to the operator and a variable factor in the process of testing. The only benefit that comes from the practice goes to the cement dealer. It takes time, is more or less expensive (if done by machine), and complicates the operations. It should be eliminated. I am of the present opinion that the paste should be mixed thin, so as to need no ramming. Say about the same consistency as it is used in practice —for Portland mortar, 1 to 1, about 17% water. I cannot see any objections to this practice, and besides getting rid of ramming, we would get about the same results which we do in practice on the works. I do not know of any machine or device that will determine the consistency of cement paste, nor do I see any need of one, because all we are aiming for is a uniform consistency. I think the most convenient way is for the Committee to specify the percentage of water to be used with each kind of cement—Portland, Rosendale, etc., neat, 1 to 1, 1 to 2, etc. This would simplify the personal equation."

Question 24.—What should be the temperature of the materials used in mixing?

The replies to this question indicate a preference for a temperature of from 60 to 80° Fahr.; a number think the precise temperature of little importance. Probably the average of the results would indicate a preference for a temperature of about 70° Fahr., when the precise temperature can be controlled.

Question 25.—What should be the temperature of the air at mixing?

The replies to this question are practically the same as those to Question 24, and indicate about the same result.

Question 26.—How should the quantity of water used in mixing be defined?

The majority of the replies to this question are to the effect that the quantity should be defined by stating the percentage of the weight of cement. Of course this percentage depends upon the character of the cement. Several of the replies indicate that the quantity should be defined simply with reference to the consistency of the mortar; thus we quote the reply of Professor Clifford Richardson: "The quantity of water to be used in mixing can only be defined as an amount suitable for making the dryest mortar which shall show water upon the surface when rammed into the mould, and which, at the same time, shall be sufficient to develop the best features of the cement, as judged by its appearance when mixed in the form of neat mortar."

Questions 27, 28, 29 and 30.—27. What should be the method of mixing? 28. Do you prefer hand or machine mixing? 29. If the latter, what machine do you prefer, or what form would you suggest for trial? 30. Do you know of any machine that has given good results? If so, what is the method of manipulation, and what are its advantages and defects?

The greater number of those who reply have only used hand mixing, or if they have used other methods, prefer hand mixing. Several, however, of those whose experience has been confined to hand mixing are of the opinion that machine mixing would be better, and that it ought to give more uniform results, but they do not know of any satisfactory machine. The machines mentioned by those who

prefer machine mixing are those of Schmelzer, Faija, Tetmajer and the "jig" used by the St. Louis Water-Works. With reference to methods of mixing, there is, apparently, not a great difference of opinion. The general reply to this question is that the material should be mixed dry, then the water added and then all worked into a paste. The majority of the members who mentioned this point advocate adding the water at once, but two especially mention the fact that they add it little by little. One adds it all at once for tension tests, and little by little for making pats. Most of the members specify the use of a trowel, although some prefer kneading with the hands, and one uses an iron spoon. We quote a few extracts which may be of interest.

Professor Clifford Richardson writes as follows: "I have found no method of mixing to compare with that done by hand, as the latter permits of some judgment in the handling of the mortar and in the amount of water which must be used. I have found that the best method of manipulation by hand is with a large steel spatula, the blade of which is 10 ins. long and 1⅝ ins. broad. This is much more satisfactory than a trowel. To the amount of cement taken for the formation of test pieces, the requisite water is added, as far as possible, all at once, and cement and water worked back and forth under the spatula and thoroughly mixed until a peculiar appearance shows that the water and cement are combined, and what appears to be a certain chemical combination between a portion of the water and the cement has taken place. In case of a slow-setting Portland cement this would require about five minutes, but with quick-setting cement, the time would be less. After an extended experience in making mortars for tests, and instructing novices in the matter, I have come to the conclusion that nothing but practice and watching those skilled in the preparation of mortar will enable a novice to acquire the necessary skill in so doing. I do not think that any written instructions can prove more than a rough guide in this respect. Perhaps, for the novice, machine mixing may be preferable to the crude attempts that he would make by hand."

Mr. C. S. Gowen writes: "We prefer hand work. After the trowel work and kneading work as noted above is done, the mass is *pressed*, not rammed, into the mould, which is then smoothed with the trowel; then immediately turned over, when the bottom or under side is in turn filled and smoothed."

With reference to machines, Captain F. V. Abbot writes as follows: "I suggest a cubical box mixer of a size to properly mix three briquettes at once. A perfectly water-tight door must be provided for the box, and this door should constitute one entire side of the box. The shaft should be continued all the way through the box, as this assists much in the complete admixture of the contents. I have not used the above machine, but I have heard that it has proved to be convenient and efficient. It should be revolved by power, and the same number of revolutions should be given to each batch."

Mr. J. L. Allison writes as follows: "Have used only Faija's, but found it satisfactory. This machine gives a thorough mixing and permits of the paste, etc., being quickly removed and of all parts being easily cleaned. The bearing of the shaft carrying the mixing blades wears rapidly."

Mr. D. Molitor writes as follows: "I would suggest a revolving cylindrical drum on a fixed axle provided with paddles or thorn-like projections as a very simple and serviceable apparatus. The ingre-

dients could be placed in the drum, the opening in drum closed, the degree of mixing could be conducted uniformly by revolving the drum a certain number of revolutions at a nearly constant rate. Perhaps the axle could be rotated in reverse direction from drum."

Mr. R L. Humphrey, who uses hand mixing, nevertheless makes the following recommendation: "The writer recommends, as a mechanical mixer, a cubical box revolving at a high speed on trunnions located on the axis passing through two opposite corners. I do not know of a machine that has yielded results that are entirely satisfactory or that are as reliable as those obtained by hand mixing."

Question 31.—How long should the mixing be continued? Should this be defined by stating the length of time, or by reference to the character of the resulting mortar?

The majority of the replies are in favor of determining the length of time of mixing by reference to the character of the mortar. This will, of course, depend upon the amount of mortar that is made in one batch.

Questions 32, 33 and 34.—32. What do you consider the best method of determining the time of setting? 33. How shall the beginning of the set be defined? 34. How shall the end of the set be defined?

The majority of the replies to these questions are in favor of the use of the standard needle as specified in the report of the previous committee of this Society. Five replies, however, indicate a preference for the German method. M. Toltz, Assoc. M. Am. Soc. C. E. writes: "We have failed to discover a method of defining the beginning and ending of the set. Have used neat cement pats on pieces of glass and timed the setting by pulling a string through the pats."

Captain F. V. Abbot writes: "I am inclined to try the method of Wm. S. MacHarg.* I have found all the needle tests very unsatisfactory in actual use, demanding extreme care on the part of the cement tester, and being greatly affected by his personal equation."

Question 35.—Should this test be made on neat cement paste, or on mortars, and if the latter, what proportions of cement and sand should be used?

A great majority prefer that the tests should be made with neat cement paste, while 4 advocate the use both with neat cement and with mortars. Mr. R. W. Lesley writes as follows: "Should be governed by the conditions under which the test is made. If the problem is merely to determine the time of the setting of a given cement, its results in sand mortar being already ascertained, the neat pats should be tested; but if the problem is to determine whether a new cement can be safely manipulated in sufficient time to allow it to be brought to the place of use on the work, then mixtures of sand and cement, in the proportions in which the mortar is to be used on the work, should be made."

Questions 36 and 37.—36. What should be the consistency of the mortar? 37. What should be the temperature of materials and of air, quantity of water, and method of mixing?

The replies to those questions are practically the same as the replies to Questions 23 and 24.

Question 38.—What should be the method of making the pats, or of filling the moulds, if they are used?

The replies to this question are in general similar to those with reference to making briquettes, and need not be specially referred to, as they are sufficiently covered by the replies to Question 48.

* *Engineering News*, Vol. xxxvii, p. 10.

Question 39.—How shall the pats or briquettes be treated during setting?

The general reply to this question is that the briquettes or pats should be kept under a damp cloth or in air nearly saturated. A number of the replies, however, simply state that they should be kept in air. Two of the replies, which state that they should be kept in air, add that they should be placed in water if the cement is to be used under water. Mr. M. J. Butler writes: "A very excellent plan is to use an iron pan inverted over pat—on the bottom of the pan, inside, fasten a couple of sheets of felt, thoroughly dampened. The set then takes place in a moist atmosphere."

Professor Clifford Richardson writes: "If the setting is determined from the briquette preserved in the moist chamber, the time of setting may, in some cases, be very much delayed. I, therefore, think that the determination of setting should be made on test pieces exposed to the air, and that the temperature should be noted, as this will seriously affect the rate of setting of all cements."

Question 40. What should be the temperature of the water in which pats are placed?

Most of the replies indicate that the pats should not be placed in water at all. If placed there, however, the temperature should generally be from 65 to 70° Fahr.

Question 41.—What do you consider the best test for soundness in the case of Portland cement? What in the case of natural cement?

Eleven replies indicate a preference for the present standard test of the American Society of Civil Engineers. A few of the replies, however, indicate a preference for the steam or hot water tests. We quote a few of the replies which may be considered typical and of interest. Professor Clifford Richardson writes: "For Portland cement, placing a thin pat of cement upon a glass plate in water at ordinary temperature for a considerable length of time. The process which I have used is that ordinarily described, and is the one which requires no elaborate apparatus. The same test is satisfactory for natural cements. Where time is an element to be considered, some of the accelerated tests I have found to be of value and have subjected pats made upon glass, and ordinary briquettes, to the action of water at 160° Fahr., or to the action of live steam for eight hours in each case, as well as the exposure of the test pieces, after they have attained hard set, to the temperature of 212° Fahr., in a dry oven. I should say that the first and last tests, representing a mild and severe form of the accelerated tests, could be carried out without difficulty in any ordinary cement laboratory. None but very inferior brands of Portland cement will fail when pats or briquettes are placed in cold water, raised to a temperature of 160° Fahr., and maintained at this temperature for eight hours. The steam tests will cause a few very satisfactory brands of Portland cement to check slightly. The dry oven, or kiln test, as it is called, lies, in severity, probably between the two. I use them both, but should not condemn a cement if it passed the 160° test satisfactorily but showed slight cracks on steaming. I would recommend that the mildest form of test alone be specified in any uniform system of tests which may be agreed upon, although the others may prove instructive in certain cases."

Mr. C. S. Gowen writes: "In either, the simplest and most practical test is by means of thin pats made with thin, sharp edge on zinc plates. These, after 24 hours in air, should be placed in water for 6 days. The bottle tests with grout can also be used. These samples should be watched for checks and cracks. A useful but not altogether

conclusive test is by means of a thermometer immersed in a grout of equal parts of cement and water. The rise of temperature, whether immediate or occurring more slowly as setting takes place, may indicate greenness or lack of seasoning. Care should be taken to have the cement and water at a uniform temperature, to place the apparatus out of draughts in a place where the temperature is as nearly as possible uniform and preferably not above 60 degrees. The dish used for this experiment should be of glass or earthenware with moderately thick sides. More elaborate tests than the above had better be referred to an analytic laboratory."

Mr, R. W. Lesley writes: "No others show any degree of uniformity or regularity, whereas the tests now in existence have borne the stress of time, and have produced results which have made Portland cement well known, the world over. In a case of this kind, where a specification is to be made governing well-equipped testing laboratories, as well as the ordinary laboratories on important pieces of engineering work all over the country, it would be a very dangerous thing, in my judgment, to attempt to lay out any line of accelerated tests, pending the recent conclusion of the German Society to maintain the present standards, and pending the report of their committee upon this subject, and especially in view of various Government engineers abandoning these accelerated tests."

Mr. J. P. Snow writes: "I consider the qualities generally covered by the term 'soundness' fully as important as the absolute tensile strength. For natural cements, our general practice is to make small pats, on glass plates, of neat cement mixed to the consistency of putty; work it thoroughly with the trowel and work up the edges of the pat so that they shall be vertical or slightly overhanging, and, as soon as made up, place them carefully in water. These pats will be from ¼ to ⅜ in. thick, and if they do not crumble down on the edges, it is taken as a sign of reliability. Another way is to work up a small pat on glass with the fingers, leaving a sort of pyramid with very sharp thin edges standing about ¾ in. high. This is carefully placed in water as soon as formed, and, unless it is the very best of cement, the edges will crumble.

"If a pat crumbles somewhat on the edges and the debris sets up so that it will hang together and not wash off, it is considered a favorable indication.

"These tests are rather severe. If a pat crumbles at the edges and the débris remains mostly friable, while the central part hardens up in good shape and shows no signs of cracking after a week's immersion, the cement may be used with confidence in its soundness. Some cements will appear to stand well for a day or two and then crack up and disintegrate. This action, if pronounced, would lead to rejection of the material.

"But little can be learned from the action of natural cement pats hardened in the air. A bluish color is taken as a good indication, but it is by no means essential. A uniform color is to be preferred to a mottled appearance, and if a decided hole of a yellowish color forms on the pat, the material is looked upon with suspicion.

"These tests, combined with those for fineness, if made by an experienced person, will indicate good and poor material as surely as tests for strength.

"It has been my experience that Portland cement does not show its good and bad qualities so readily as natural cement when tested in pats. Good Portland cement will quite frequently crumble down when

put into the water as soon as mixed, and if allowed to partially set in air before immersion it would be a very poor article indeed that would not show up all right. The crumbled débris of a pat will set quite hard and firm if the cement is good, much harder than in the case of natural cements. The color of a pat set wholly in the air and sunlight is a more valuable guide, I think, in Portland than in natural cements. The boiling test, so-called, is useful in ascertaining if Portland cement has an excess of lime, although it has not been used much by us."

Mr. R. L. Humphrey writes: "Numerous tests have been devised for determining the soundness of a cement, or more properly, its constancy of volume, without as yet resulting in a test that is thoroughly reliable. Probably the German requirement that cement, after hard set, immersed in water maintained at a normal temperature, shall show no signs of swelling, checking or disintegration, is the most reliable, both for natural and Portland cements. A cement, however, unless of very bad quality, rarely fails to meet this requirement in short periods of time. As a quick test for the determination of the soundness of a cement, it is therefore of no practical value. Regarding the accelerated tests for determining constancy of volume of Portland cement, it is the writer's opinion, based on numerous tests, that the best method is the 'hot water' test. In this test the cement is made into pats 2 or 3 ins. in diameter, ½ in. thick at the center, with thin edges; these pats are preserved in moist air for 24 hours and then placed in water maintained at a constant temperature of 175° Fahr. These pats should show no signs of swelling, checking or disintegrating, and as a rule should adhere firmly to the glass on which they are made. The apparatus used by the writer for the hot water test is described on pages 12–13–14 of the accompanying pamphlet; it is too elaborate for any except a well-equipped laboratory. The hot water test can be made very readily in any vessel in which water can be boiled, although the results of the tests made in boiling water are not as a rule as reliable as those made in water maintained at a constant temperature of about 175° Fahr. A thoroughly normal Portland cement will meet this test. For natural cement, this test is too severe, and only the best grades will stand it. Pats made from natural cement and preserved in water at normal temperature, and in air, afford more reliable results. For the determination of the soundness of a cement, the writer preserves pats of neat cement, both natural and Portland, of normal consistency in steam as soon as made, in air, in water at normal temperature, and in hot water, at the end of 24 hours in moist air. Pats preserved in air furnish considerable information. A good normal cement should have a uniform color, should remain firm and hard, and show no signs of swelling or checking. Like the cold water test it requires time and is not therefore adaptable for quick tests."

Question 42.—What proportions of cement and sand should be used in mortar for tests of tensile strength?

The general opinion in reply to this question is in favor of the use of the proportion of 1 to 3 for Portland, and 1 to 2 for natural cements. A number of members mention other proportions, and a number are in favor of making only neat tests. We quote the reply of Mr. J. L. Allison as interesting in connection with the last-mentioned opinion: "It is my belief that tests with sand, as forming part of the acceptance tests, are not only unnecessary, but undesirable. The object of tests is to secure a cement of good quality in a state most suitable for making mortar. The tests for specific gravity, soundness, tensile strength, etc., show the quality; and the testing by means of fine sieves for fine-

ness of grinding, shows its suitability for making mortar. I have never known a good cement, if finely ground, fail to give a high test with sand, while on the other hand, I have known poor cements, when finely ground, to give a high mortar test. The undesirable feature of the test is that it introduces a dual standard of strength. If the tensile strength of a cement should fall below the specifications—while the sand test passed—a contractor would have (under certain conditions) a chance to force the acceptance of a weak cement. If this test is to be included in acceptance tests, the specifications should, in my opinion, distinctly state that the results obtained from it would not be taken as giving any information as to the *strength* of the cement; but would be considered only as bearing on the question of the fineness of grinding. It was at one time my opinion that tensile tests should be made on mortar briquettes only; but a close acquaintance with the differences in results caused by slight differences in mixing and filling, forced me to abandon that opinion. Information of much value may be obtained from mortar tests made for the purpose of determining the relative value of different sands, and for finding out the extent to which the sand to be used may be added to the cement for the different classes of work. For this purpose the mixing should approximate to the actual conditions on the work, and the mortar should be thin enough to flow in the mould under the influence of the point of a trowel moved edgewise. No compression whatever should be used in filling."

Question 43.—Do you advocate adhering to the Am. Soc. C. E. form of briquette in future requirements? If not, what form do you prefer?

A large majority of those replying to this question are in favor of adhering to the present form of briquette adopted by the Society, 25 replies indicating this preference, while four advocate a change. Of those preferring the present form, one thinks the clip should be modified, while Mr. R. L. Humphrey has modified the form slightly by rounding the corners. We quote a few replies. Professor A. V. Sims writes: "Have not been able to obtain satisfactory results with the Am. Soc. C. E. form of briquette, nor with any form of briquette when broken by any clips on the market, nor with any form of briquette or clips with hand-made briquettes." He has been experimenting with a new form of clips and a briquette machine of his own invention and a new mould.

Professor J. B. Johnson writes that he is satisfied that the form of briquette should be changed to something like that shown in Fig. 356 of his book on "Materials." This shows 25 to 30% greater strength than the common forms, either American or German.

Professor Clifford Richardson, on the other hand, writes as follows: "I can see no reason for departing from the form of briquette recommended by the American Society of Civil Engineers in 1885. When properly used it has given entirely satisfactory results. If any departure should be made, it should, in my opinion, be in the direction, not of an alteration of the shape of the briquette, but of reducing its size to that of the cross-section employed on the Continent, 5 sq. cm., or 0.775 sq. in. Briquettes with the latter cross-section require less mortar for their preparation and can be more satisfactorily made. They will also give considerably higher results in case of tensile strength."

Question 44.—Is your preference based on comparative experiments, or is it the result of satisfactory experience with one form?

Of the 25 who are in favor of retaining the present form, 18 state their opinion to be the result of satisfactory experience with this form.

Four state that their opinion is based on comparative experiments, and 3 state that their preference is based on both comparative experiments and satisfactory experience with the present form.

Question 45.—Upon what sort of surface should the briquette be made?

The practically unanimous reply to this question is that the surface should be non-absorbent and smooth; that is to say, a surface of marble, slate, glass or smooth metal. One of the German manufacturers in his reply to the circular gives naturally, of course, the reply that it should be "a smooth metal surface, covered with a piece of moistened blotting paper."

Question 46.—Should the briquette be finished with a trowel on both sides?

Thirteen replies are that the briquette should be finished with a trowel on both sides, and 16 that it should be finished only with the trowel on top. Some think that the bottom which rests on a smooth surface will be smooth and completely filled in any case, but others, like Professor Clifford Richardson, think differently. Thus Professor Richardson writes: "Compressed upon both sides; have used for this purpose a stiff putty knife which has been ground to a proper width to pass the narrowest section of the mould. If the briquettes are not compressed upon both sides, I have found that they have a very uneven density."

Question 47.—What should be the consistency of the mortar?

The replies to this question agree substantially with the replies to Question 23, and need not be further dwelt upon. Mr. R. L. Humphrey gives the following formula: "The consistency of the mortar is best regulated by determining the percentage of water required for different proportions of sand. The writer has adopted the following formula for this purpose: $E = \frac{2}{3} N A + 60$, where

N = weight (in grams) of the water required for 1 000 grams of neat cement.

A = weight of cement (in kilograms) in 1 000 grams of sand mixture.

E = weight (in grams) of water required for the sand mixture."

Question 48.—What method of filling the moulds do you advise? Do you recommend the use of a machine for moulding, and, if so, what form would you suggest?

Seven replies indicate a preference for machine moulding, while 21 indicate a preference for hand moulding. Of those preferring the machine, two prefer the Bohme hammer; one suggests a machine built like the old-fashioned hand press used in re-pressing brick; two prefer a machine, on general principles; one (Captain F. V. Abbot) says: "Fill the mould in several layers, pressed down with the thumbs. Have a considerable surplus. Fit a second mould over the first, put a brass plunger in the top mould, and apply a steady pressure of 500 lbs. to the top of the plunger. Keep this pressure on the briquette for two minutes exactly." Another, Professor Richardson, writes: "For ordinary tests I would recommend filling the mould by hand, as this can be done much more rapidly than with any machine except that devised by Professor Jameson, of the State University of Iowa, which, however, is not satisfactory for use with standard quartz mortar, which is the mortar most generally used. I should recommend the use of a machine for moulding, such as that employed at the Royal Testing Laboratory at Berlin, where absolute tests of any cement are to be made; but for ordinary relative tests ordinary hand moulding is quite as satisfactory and much more rapid."

We quote a few more of the replies. C. D. Purdon, M. Am. Soc.

C. E., writes: "I use a putty knife or small trowel, filling the mould about one-quarter or one-fifth and tamping with the handle. Have seen a machine by William Lee Treadwell, M. Am. Soc. C. E., which I think would give good results."

D. M. Andrews, M. Am. Soc. C. E., writes: "The cement should be thoroughly pressed into the moulds with the trowel or fingers, and all air bubbles worked out, but should not be rammed."

Mr. J. L. Allison writes: "The moulds—40 mm. deep—resting on glass slabs, should be filled without compression (see Question 23). The moulds should be given the slightest possible film of oil and laid on a glass slab. The paste should be made to flow to all parts of the moulds by the edgewise motion of the point of a trowel held vertically. The surplus paste should be removed, and the surface smoothed with the trowel, with the least possible amount of compression or work on the paste. The moulds, when filled, should not be lifted or otherwise displaced from their first position on the glass slab until the briquette becomes hard set. I am decidedly opposed to the use of a machine, unless it could be arranged so as to fill the moulds without putting compression or other work on the paste."

Mr. A. O. Graydon writes: "We fill moulds by hand. We place sufficient cement at once to fill mould about 1 in. over the top, then run point of trowel through several times to take out air bubbles and stroke off, using only light pressure of the hand in doing this. Do not favor a machine."

Professor J. A. L. Waddell writes: "I advise filling the moulds by hand as previously described. I have experimented, at considerable expense, upon a machine for making briquettes. It appeared at first to do the work very satisfactorily, but I found in the long-time tests so much irregularity in the strength of the briquettes that I was forced to abandon all the tests of briquettes made by the machine. I know of no satisfactory machine for making briquettes."

Mr. R. W. Lesley writes: "The best method of filling the moulds is to take the material from the plate or wash-basin in which it has been mixed, and to place it in the moulds with the fingers or with a spatula, and then to press it in until the cement is well and equally distributed all over the mould. So far as machines for making briquettes are concerned, I know of none except that used by Tetmajer, and also the German machine already referred to in my replies to Questions 28 and 29, and, as stated, the Tetmajer machine is much the more rapid, and the German machine has fallen into disuse in several cases where such apparatus has been used."

Mr. R. L. Humphrey writes: "After thoroughly mixing the neat cement or the mortar by hand and trowel, the mixture is placed in the mould and firmly pressed in with the fingers, without ramming, and the mould smoothed off on sides with trowel."

Question 49.—Have you used the machine you suggest, and have the results been satisfactory?

Of those recommending a machine, 4 have used those which they recommend, while three have not. Of those recommending hand mixing, two have tried a machine with poor results and know of no satisfactory machine; while 10 have not tried a machine, and 8 do not answer Question 49. The replies of Professors Richardson and Waddell to Question 48 are interesting in this connection.

There is no doubt that the consistency of the paste and the method of filling the moulds are important factors in the problem of securing uniform results.

Questions 50 and 51.—How should the briquettes be treated during the first twenty-four hours after moulding? How should they be treated during the remaining time until tested?

The majority of replies to this question indicate that the briquettes should remain in the moulds till set, then be taken out and placed upon some non-absorbent surface and covered with a damp cloth, or else placed in a box exposed to moist air and allowed to remain for 24 hours, after which they are to be immersed in water. Several keep the briquettes in the moulds until they are ready to be immersed in water, and a few do not immerse in water at all, but preserve in air until tested. One or two call attention to the fact that the briquettes should be tested immediately after being taken from the water. Mr. T. C. Hatton states that he has found considerable difference in the tests resulting from leaving the briquette in the air some time after being taken from the water as compared with tests made immediately after the briquette is removed from the water.

Question 52.—If placed in water, how often should the water be renewed, and is it important that it should be maintained at a nearly constant temperature? What should the temperature be?

There is considerable difference of opinion with reference to this question. Fifteen replies state that a constant temperature is important, while 6 state that it is unimportant—one of these considering that it is important only during the first week. With reference to the temperature, all those who specify practically unite in stating 65 to 70° as the proper temperature.

With reference to renewal of water, 10 state that it is unimportant, while 15 consider that it is important to renew it. A few think it should be renewed daily, if possible; 5 that the briquette should be immersed in running water; one thinks it should be renewed every three days; one every week at least, and one when it becomes greasy or slippery.

Professor J. A. L. Waddell writes that he renews the water two or three times in the first two or three weeks, especially if it begins to look cloudy, and after that he simply replenishes the water when it evaporates, unless it should have a dirty appearance. One of the German manufacturers states that it is not necessary, either to renew the water or to keep its temperature constant, but in the case of short-time tests, it is essential to maintain a uniform temperature.

Questions 53 and 54.—At what age should briquettes be broken for acceptance tests on ordinary work? Under what conditions would you deem it essential to make longer time tests?

It is difficult to give a summary of the replies to this question, but the general opinion seems to be that the test should be made at 1 day and 7 days. A good many specify 1 day for natural and 7 days for Portland, while others, who do not specify, probably mean this. A good many name longer times, but it is not to be presumed that long-time tests would be required for the acceptance in the field of well-known brands. In answer to Question 54, a good many specify that longer time tests should be required where the shorter-time tests are not satisfactory or are suspiciously high, or in case of new or unknown brands of cement.

Question 55.—Will weighing briquettes before testing give information of value, and, if so, what?

To this question, 10 members reply "no"; 5, "yes," and 3 are uncertain. We quote several of the replies. Professor Clifford Richardson writes: "Weighing briquettes, not before testing, but immedi-

ately after they attain their hard set, is, I believe, one of the best guides in securing uniformity and satisfactory results in making test pieces for neat cement and of sand mortar. It is also the most excellent check for novices to use in acquiring the requisite skill. For example: with a German Portland cement, Germania Brand, it was found that a briquette made in one of Riehlé Bros.' moulds of the pattern recommended by the Am. Soc. C. E., when made of neat mortar, weighed from 140 to 141 grams, and had a density, calculated from the volume of the briquette, which is, as near as can be ascertained, from 66 to 66.5 cu. cm., 2.136 to 2.121, while cubes made in the same way for crushing tests had a density, 2.229 to 2.121. Briquettes made of mortar with 3 parts of standard quartz to 1 of cement, weighed from 130 to 125 grams, and had a density of 1.970 to 1.889, while the crushing briquettes had a density of 2.022 to 1.945. In the case of sand mortar the difference in the weight of the tensile strength of the briquettes reached as much as 5 grams, which is too great. The lightest briquettes in this case would be thrown out, and a difference of not more than 3 grams at the outside allowed. As samples of the density and weight of briquettes of the pattern recommended by the Am. Soc. C. E., the following figures, determined with various cements, will serve.

DENSITY OF BRIQUETTES.

	Tensile.		Crushing.	
	Neat.	Sand.	Neat.	Sand.
Rosendale	2.06	2.02	2.07	2.07
Utica	1.95	1.96	1.98	2.25
Double Star	1.86	1.94	2.12	2.28
Mankato	1.95	1.94	1.95	2.05
Germania	2.14	1.97	2.23	2.02
Vulcanite 25%, 10% water	2.15	....	1.97	....
Iron Clad	2.18	1.96	2.21	2.18
Brooks and Shoobridge 19% and 10% water	2.30	1.89	2.28	2.12

A briquette should weigh:

Portland 20% and 10% water	142	130	290	280
Portland 19% " 9% "	144	132	299	....
Rosendale 28% " 14% "	136	133	272	272
Utica 32% " 14% "	129	130	257	296
Double Star 38% " 15% "	123	140	254	299
Mankato 30% " 14% "	129	129	255	269

"I fully believe that a recommendation, at least, should be introduced in any rules for uniform testing of cement, that all briquettes should be weighed and the densities given with the results of the physical tests."

In connection with this reply from such a well-known authority, it is interesting to note, as showing the difference of opinion between those best qualified to judge, that Professor S. B. Newberry, an eminent chemist and a man experienced in the manufacture of cement, replies "no" to this question. Mr. R. L. Humphrey writes: "The weight of a briquette, or, better, its specific gravity, as a means of determining its density, is a valuable aid in forming an opinion as to the

merits of the briquettes in comparative tests, or where they are made by different persons."

Question 56.—What form of clip do you prefer?

The replies to this question generally indicate a preference for the Fairbanks or Riehlé form of clip with rubber bearings. Three replies are that no form is satisfactory, and five simply state that they prefer the Am. Soc. C. E. standard. Mr. J. L. Allison, whose reply is included in the numbers given above, writes as follows: "The clip with rubber tips furnished with the Riehlé machine. I have not used any other form of cushioned clip. With a small Fairbanks machine (1 000 lbs.) strong briquettes were *generally* broken at the points of contact of one of the clips instead of at the smallest section. In order to compare the machines, an extensive series of parallel tests was made—one-half of each set of briquettes being broken on each machine. The general average of the results gave the following ratio:

"Strength as determined by Riehlé machine (cushioned clips)	1.00
Strength as determined by Fairbanks machine (uncushioned clips)	0.77

"With the former machine the breaks occurred at or near the smallest section; it is probable that the readings of the machine gave a close approximation of the actual tensile strength of the briquettes. With the latter machine, however, the briquettes were not pulled apart, but were broken by some kind of crushing, or splitting, action at the points of the clips, where the pull of the machine represents only the vertical components of the total pressures concentrated on lines or points of contact."

We quote a few other replies. E. B. Noyes, M. Am. Soc. C. E., writes: "The clip should conform to the briquette for, say, ½ in. This would necessitate a device to prevent twisting the briquette by a side strain. My experience with the Am. Soc. C. E. clip shows that, the stronger a briquette is, the more likely it is to break centrally. With weaker cements, the break is more likely to be at the point of the clip, showing some wedge action."

Mr. D. Molitor writes: "I believe that the principal difficulty experienced with clips is not so much the shape of the sample and of the clips, but the manner in which the clips are held by the jaws of the machine. By giving this connection a universal motion so that there will be absolutely no bending moment on the sample, it is fair to expect more uniform results."

Question 57.—What should be the distance between opposite gripping points of the same clip?

The replies to this question vary all the way from ¾ in. to 1¼ ins. There are only 13 definite replies.

Question 58.—What should be the rate of applying the stress?

The majority of the replies indicate a preference for a rate of about 400 lbs. per minute. Some say that the stress should be applied "slowly," while several state that the rate is not important as long as it is uniform. One or two call attention to the fact that with strong cements one-half or three-quarters of the load may be applied rapidly, but that after that, the rate must be uniform and slow.

Question 59.—What style of testing machine do you prefer?

Sixteen replies indicate a preference for the Fairbanks machine, while three prefer the Riehlé, one the Olson, two (including one German manufacturer) the Michaelis machine, and three either Fairbanks or Riehlé. D. M. Andrews, M. Am. Soc. C. E., sends a blue print of a

proposed machine, while one or two emphasize the fact that the stress should be applied by power rather than by hand, to insure regularity.

Question 60.—Can you suggest any desirable modifications to machines now in use?

Most of the definite replies to this question indicate that the trouble in present machines, if any, is in respect to the clip bearing. Professor S. B. Newberry says that the machine should be larger, to give more accurate results with high strength. Several members suggest the use of a small electric motor to run out the weight and apply the stress uniformly. Several other suggestions are made which it is not necessary to quote.

Question 61.—What special precautions are necessary in breaking briquettes?

The majority of replies to this question indicate that the principal precaution is in adjusting the clips. Quite a number, however, call attention to the necessity of avoiding any sudden application of stress. Several others also call attention to the necessity, already alluded to, of breaking the briquettes immediately after taking from the water.

Question 62.—Do you advise compressive tests, and, if so, why?

The majority advise compressive tests, although several do not consider them generally practicable, except in a well-equipped laboratory. Professor J. B. Johnson states that there is "no necessity for tensile or cross-breaking tests." Professor Clifford Richardson writes: "I am very much in favor of tests of compressive strength of any cement and mortar where the facilities are available for carrying them out. The results of these tests correspond more closely to the conditions existing in practice, and reveal certain characteristics which are not shown by tensile strength."

Mr. R. L. Humphrey writes: "Suitable machines for making compression tests are expensive, require considerable space, and are not, therefore, adapted for temporary testing laboratories. They are desirable, however, and should be made in all permanent laboratories. As already stated, neat cement becomes very brittle at the end of six months or longer periods of time, and when tested in tension snap off before the full strength is reached, giving rise to the impression that the cement is losing strength. Whereas the same cement tested in compression would show no decrease, unless the cement be of very poor quality; but, on the contrary, would show an increase of strength."

Most of those advising compressive tests call attention to the fact that the cement is really used in that way, and that there is no necessary relation between tensile and compressive strength.

Question 63.—What form and dimensions of test piece do you prefer?

The replies to this question recommend a cube varying from 1 in. to 12 ins. The replies, however, are few and give little means of judging of the opinion of the Society.

Question 64.—Should the test piece be treated differently as regards manipulation of mortar, mixing or setting, from tensile specimens? If so, please state in what particulars, and why?

The replies to this question are practically unanimous that there should be no difference in the manipulation. Two, however, consider that it would be necessary to use the Hammer apparatus in forming the cubes. We quote two replies. Professor Clifford Richardson writes: "Test pieces for the determination of compressive strength are much more difficult to make than those for tensile strength. The

mortars, however, should be prepared in the same way. It would seem that some sort of machine moulding is more necessary for the preparation of compression test pieces than those for tensile strength. The danger in hand moulding lies in the possibility of the material lying in the test pieces in unequal layers which are, at the same time, unequally bound together."

Captain F. V. Abbot writes: "The cubes should be made of concrete taken directly from the batches on the work. They should be rammed and otherwise treated as nearly as possible like the concrete in the actual work. This gives the best available indication of the material actually constituting the structure. Such cubes should be kept in damp sand 28 days and then be broken."

Question 65.—How should the specimen be prepared for the testing machine?

In answer to this question, the suggestion is generally made that the sides of the compression piece to which the force is to be applied should be either carefully trued or trowelled, or that lead plates, or thick paper, or plaster of Paris, or cardboard, or a layer of very fine sand, should be placed upon these surfaces. One member replies that the test piece should be pressed into the moulds, but not rammed. One member suggests that the ends to which the pressure is to be applied should be allowed to set in contact with glass plates so as to insure perfectly even surfaces.

Question 66.—What form of testing machine do you recommend?

In reply to this question, mention is made of the Fairbanks, Riehlé and Amsler Laffon machines.

Question 67.—What should be the rate of applying the stress?

The replies to this question indicate generally the same rate of application as for tensile stress, but several replies indicate a more rapid application, such as 1 000 lbs. per minute.

Question 68.—Do you advise bending tests? If so, under what conditions and why?

In reply to this question, there are 6 replies of "no," and 7 of "yes." Two replies are only for concrete, and two only for research laboratory.

Mr. J. P. Snow, whose reply is typical of several others, writes: "I think that the nature of cement can be ascertained as well by transverse as by tensile tests, and that an outfit for making the former can be got up much cheaper than for the latter. Tests could be made more rapidly, and there would be no uncertainty as to the directness of the application of the load. I see no need for making both transverse and tensile tests."

Question 69.—What form and dimensions of test piece do you prefer? What span?

The replies to this question are quite diverse, and it is not necessary to summarize them.

Question 70.—Should the test piece be treated differently as regards manipulation of mortar, mixing and setting, from tensile specimens? If so, please state in what particulars, and why?

The general reply to this question is "no."

Questions 71 and 72.—What form of testing machine should be used? What should be the rate of applying the load?

The replies to these questions are so few that it is not desirable to summarize them.

Miscellaneous.—Under what conditions do you consider the tests indicated below necessary or desirable? What should be the manipulation for the test

if used? I.—Adhesion. II.—Abrasion. III.—Resistance to freezing. IV.—Resistance to action of sea water.

Under this head, a number of suggestions are made by various members, which we quote:

S. B. Newberry, Assoc. M. Am. Soc. C. E., writes: "These tests involve too much extended experimenting, in my judgment, to be of value in testing individual cements."

C. A. Miner, Assoc. M. Am. Soc. C. E., urges that the Committee establish two standard methods of testing, the laboratory test and the field test. The former should embody all the refinement deemed necessary and only possible in the well-equipped testing laboratory. The latter should be so simple that it could be performed at any time. on short notice, with simple and crude apparatus, most of which could be made by a fair mechanic, and the total cost of which would not exceed a few dollars.

Mr. A. S. Cooper thinks that the principal cause of discrepancy is in the amount of water used. The standard adopted by the Am. Soc. C. E. is liable to different interpretations by different operators A "stiff paste" may mean many different things. The "consistency of moist snow" is also uncertain. He believes the German standard is nearest correct, that water must just appear on the surface after 100 blows with the Bohme hammer, but this, too, is not satisfactory, and gives no true standard for comparing different cements. He considers the German machine clumsy and slow. He thinks some machine for moulding briquettes under a high and uniform pressure, say 3 000 lbs. per square inch, would give good results.

Professor A. V. Sims is satisfied that he has a method which will give uniform results.

E. S. Gould, M. Am. Soc. C. E., believes in making up a batch of mortar, enough for 10 to 20 briquettes, moulding half into briquettes and immerse in the usual way, letting the remainder stand till it has taken a decided set, then breaking up and moulding. He believes a good Portland cement stands the test well, the broken-up mortar giving strength sometimes equalling the first briquette. He allows the use of Portland cement mortar which has stood the whole day on the mortar board, thoroughly re-tempering it and working it over.

Mr. T. C. Hatton writes: "III.—When the cement work is intended to be exposed to the action of the water and atmosphere intermittently throughout great changes of temperature, it is wise to make comparative tests of the several cements offered. A short time back, I had to build two concrete walls exposed to tidal influence and ice. The briquettes were mixed as usual and were placed in water with 60° temperature, which was gradually reduced to 32°, and so maintained for the allotted time, 23 hours, 61½ days and 27½ days. They were then taken out and immediately immersed in boiling water, where they remained for 1 hour, 12 hours and 12 hours, respectively, at the end of which time they were taken out, wiped dry, and immediately tested, and the cements which did not lose over 5% of their standard tensile strength were accepted.

"IV.—Should be left to the judgment of the engineer. Not necessary to specify when establishing a standard.

"I.—The following is a simple adhesion test I daily use upon my rounds in the field, for no particular reason except to see that the mortar has been properly measured and mixed, and it will detect any weakness in this respect very quickly. Take two bricks, after being wet, rub them together in the fresh mortar until their inner surfaces are entirely

covered with the mortar, and then place them in the air, one on top of the other, where they will be undisturbed, and after three minutes raise the mass by taking hold of the top brick; if the mortar holds the lower brick to the top one, the mortar may be considered properly mixed."

Professor J. A. L. Waddell writes: "There are two other tests which I make occasionally, concerning which you have asked no questions, so I will give you a description of how I make them. They are the rise-in-temperature test and the re-tempering test. To make the first, I mix 5 oz. of cement with 1 oz. of water, then insert the bulb of a thermometer in the mixture and note the rise in temperature. This, I find, runs from zero to as high as 13°, the time ranging from 4 to 5 minutes. The greater the rise in temperature, the greater is the probability of there being too much free lime or magnesia in the cement. Nevertheless, I have known the temperature to rise considerably in cements that gave no other indications of weakness whatsoever, and which were eminently satisfactory in respect to strength, etc.; consequently I do not rely very much upon the rise-in-temperature test.

"The re-tempering test I make as follows: Four sets of briquettes are employed. In the first set, the mortar is mixed and put into the moulds at once in the ordinary way; in the second, it is mixed continuously for 30 minutes and then put into the moulds; in the third, it is mixed continuously for 1 hour and then moulded; and in the fourth, the mixing is continued for 90 minutes. Often, however, I omit the 90-minute test because of the time and trouble involved thereby. Water is added during the mixing as is required to keep the mass plastic. The briquettes thus made are tested at 7 days, 1 month, and 2 months, and a comparison of the strength for sets 1, 2, 3 and 4, will indicate the extent of the injurious effect of the re-tempering. It might be advisable to prepare enough briquettes to carry these re-tempering tests to a longer period of time."

Mr. E. B. Noyes writes: "III. In several years' experience on the New York State Canals, work was continually laid in freezing weather, sometimes even at 0° Fahr. Salt was used freely. I do not remember to have seen mortar injured more than 1 in. from the face of the work. It was usually required to rake out and point after the end of freezing weather in spring."

Mr. M. Toltz writes: "III. As we are often compelled to build masonry in winter time at a temperature of from zero to 30° below zero, the experience we have had is about as follows: The mortar is mixed hot; that is to say, the water is heated and sand is heated. The mortar freezes in the work, but thaws out again in the spring and sets to our satisfaction. We never had, as yet, any trouble in regard to freezing. We have never used any salt to put in the water to prevent its freezing."

M. L. Holman, M. Am. Soc. C. E., suggests that the Committee divide the subject into two parts, viz.:

a. The testing of cements of known reliability.

b. The testing of new cements.

He adds: "Allow me to further suggest that the Committee take the requisite time to gather up all available information, and, if necessary, make some experiments."

Mr. C. S. Gowen writes: "IV. The testing of cement should be confined to simple processes with simple apparatus, in order that it may be done by the engineer at his field of work *upon the delivery of the cement in question*. In this way only can the engineer satisfy himself in regard to the actual thoroughness and reliability of the testing work

done. It seems to me that to go much beyond the various processes outlined in the answers stated above is for the engineer to encroach upon the field of the analytical chemist or the laboratory of the cement manufacturer, in neither of which places is he in a position or in circumstances to properly compete, or even to follow processes. With, however, simple methods in use at his own office he can take the time to follow them, and to assure himself that the testing is properly done. He can protect himself against poor cement always, even if he does at times reject some that is good which may have offered some doubtful phases to him."

Mr. R. L. Humphrey writes: "I have not adopted mechanical methods for mixing and moulding because (1), they are too slow for rapid work, and therefore unsuitable for use, except in purely experimental laboratories, and (2) the results are no more reliable than those obtained by hand mixing. The writer has observed that the greatest objection to the American Society rules of 1885 is the difficulty in interpreting just what constitutes a 'stiff plastic paste.' The wide variations in the results of different experimenters is due largely to this cause.

"As the most important tests of the quality of a cement are affected by the consistency of the paste, it seems to the writer that an accurate and reliable method for determining the normal consistency is the first essential consideration. In the formulation of specifications there should be a distinction made between laboratory and field testing, or testing on the site of the work in which the cement is to be used. It appears possible to adopt such a system as will be applicable to both classes of testing.

"Adhesion and abrasion tests, and the tests of the resistance to freezing, and the action of sea water, are only desirable under special conditions in which the information they may furnish has an important bearing on the use of the cement. As tests for the reception of material they are not of special value.

"Permanent laboratories should make tests of this character, and contribute their results to the available data on these subjects.

"In conclusion, the writer begs to suggest that the Committee select from the answers to their circular letter those methods which give promise of the best results. Having made this selection, he recommends that the Committee conduct a series of experiments, using the proposed methods and endeavor, if possible, to determine which method yields the most satisfactory results. The Committee would then be better able to adopt definite conclusions."

Mr. W. H. Broadhurst writes: "I would especially draw the attention of the Committee to the necessity of a uniform treatment of the briquette after moulding, *i. e.*, whether the briquette should be exposed to the air of the laboratory until 'hard set' or whether the briquette should be covered with a damp cloth as soon as moulded. I think that difference in manipulation at this point is one cause for non-uniformity in results with different operators. I would also draw the attention of the Committee to the fact that in many laboratories where a moist chamber is not at the disposal of the operator, the briquettes are covered with a damp cloth and subsequently with an oil cloth to prevent evaporation of the moisture in the cloth. This has a tendency to increase the temperature, and hence leads to non-uniformity of results."

Mr. E. S. Gould considers the principal object to be to obtain uniformity in a cement of established reputation. He would confine

himself to a cement of established reputation and by making frequent tests with his own hands would make sure that he was not receiving deteriorated material. Before adopting any new cement, he would test it against a cement of established reputation, making all tests himself, and in the same manner. He would also use the new cement tentatively in the work, in places where he could see its behavior, and then after a few years, he would be willing to express an opinion of its merits.

He would also test the sand to be used competitively with sand that he knew to be good, using the same cement. He would require no specified strength.

He does not believe in American natural Portland cements, having found (when he used them eight or ten years ago) little uniformity in them, and believing that too many different qualities are made. He had found by accident that some of these cements when allowed to stand in the air without being covered with a damp cloth, went to pieces, some at the end of only a few months, while immersed briquettes of the same cement behaved satisfactorily. He thinks that any hydraulic cement that does not almost immediately go to pieces under water will always improve by prolonged immersion, and the question is how it will behave when not immersed, and kept entirely away from moisture.

He has always found foreign Portland cements to keep on increasing in strength when not immersed, though not as fast as when immersed.

He believes that it will be impossible to fix an exact standard by which the results of different operators can be compared, and thinks it makes little difference how the tests are made provided each tester always works in exactly the same way.

He feels sure it is only the sand test that is worth anything; for one reason, because in a neat test, the coarsely ground cement always shows better than the same cement finely ground. He thinks that in a competitive test of cement, the highest possible dose of sand should be used. He does not believe that the requirement that no cement shall be used till tested is ever carried out.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(March 14th to April 11th, 1900.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c
(14) *The Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City, 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels, Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*. Stockholm, Sweden.
(57) *Teknisk Ugeblad*. Christiania, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

LIST OF ARTICLES.

Bridge.

Mason Work On the New East River Bridge Anchorages.* (14) April 7.
Repairing a Fractured Stone Arch Bridge, P. & R. Ry.* (18) March 31.
Transfer Bridges.* (11) Serial beginning Feb. 23, ending March 23.
Rolling Lift Bridge at the Boston Terminal (N. Y., N. H. & H. R. R.)* (13) March 15.
Pont à Transbordeur de Rouen.* M. F. Arnodin. March.
La Passerelle Métallique d'Oberschönweide sur la Sprée, près de Berlin.* (33) March 3.
Ponts à Bascule Roulants (Système Schertzer).* (36) March 10.
The Alexandre III Bridge, Paris.* J. Guenaire. (46) March 10.

Electrical.

The Possibilities of Wireless Telegraphy.* (42) Dec., 1899.
The Action of Electric Tramway Currents on Submarine Telegraph Cables and Other Electric Circuits.* Prof. Andrew Jamieson. (26) Serial beginning Feb. 23, ending March 9.
Dynamo Testing and the Care of Dynamos. Norman C. Woodfin. (47) Serial beginning Feb. 24, ending March 17.
Some Mechanical Details of Dynamo Design.* E. Kilburn Scott. (26) Serial beginning March 2, ending March 16.
The Controller On Electric Motors.* (48) March 3.
The Designing of Electrical Generating Stations. (26) March 9.
The County of London Electric Lighting Company's Electricity Works.* (26) Serial beginning March 9, ending March 16.
On Electric Lighting Cable Breakdowns.* Gisbert Kapp. (26) Serial beginning March 9, ending March 23.
The Electric Process of Annealing Armor Plate.* (22) March 16.
The Pollak-Virag System of Telegraphy.* Chas. H. Garland. (26) Serial beginning March 16, ending March 23.
Wallingford, Conn., Municipal Electric Lighting Plant.* Thomas C. Perkins. (27) March 17.
Fundamental Ideas of Alternating Currents.* Prof. Dugald C. Jackson. (27) Serial beginning March 17. ending March 24.
New Electric Locomotives.* (48) March 24.
The Regulation of Storage Battery Voltage. Roderick Macrae. (27) March 24.
The Tariffville Plant of the Hartford Electric Light Company.* (14) March 24.
Greenock Corporation Electricity Works.* (26) March 30.
The County of London: Electric Lighting Company.* (27) March 31.
Electric Power Distribution: Works of the Westinghouse Air Brake Company.* (25) April.
Incandescent Lamps.* Francis W. Wilcox. (3) April.
Electrical Ignition for Gas and Gasolene Engines. P. P. Mungesser. (27) April 7.
Installation Hydro-Electrique des Chutes du Niagara: Nouveaux Développements.* G. H. (33) March 3.

Marine.

Ships on the Wolga.* R. Weis. March 3.
The Saloon Steamer *Kaiserin Augusta Victoria*.* (48) March 24.
Krupp Central-Pivot Gravity and Spring-Return Carriages for Naval Ordnance.* (19) March 31.
Engineering in the United States Navy: Its Personnel and Matériel.* Rear-Admiral George W. Melville. (10) April.

Mechanical.

The Manufacture of Calcium Carbide as Related to the Iron Industry.* Liebetanz. (50) Serial beginning March 1, ending March 15.
Improvements in the Longworth Power Hammer.* Ernest Samuelson. (22) March 2.
Liquid Air.* F. Walter. (53) Serial beginning March 2, ending March 9.
A Coke-Oven Plant for the Supply of Illuminating Gas.* F. Schniewind. (22) Serial beginning March 2. ending March 16.
On the Mechanical Theory of Steamship Propulsion.* Robert Mansel. (12) March 9.
New Forms for Gear Teeth.* G. Lindener. (48) March 10.
Test of a Special "Porter" Governor.* (47) March 10.
Large Gas Engines.* E. Meyer. (48) Serial beginning March 10, ending March 17.
Repairing a Broken Fly Wheel.* James M'Bride. (20) March 15.
An Experimental Investigation of the Thermodynamical Properties of Superheated Steam. John H. Grindley. (12) March 16. (47) March 24.
Condensation of Steam in Blower Systems of Heating.* Prof. R. C. Carpenter. (47) March 17.
The Distribution of Energy on its Way Through the Steam Engine.* (47) Serial beginning March 17, ending March 24.
Friction of Steam Packings.* Charles Henry Benjamin. (20) March 22.
Pneumatic Mail Despatch.* (62) March 22.
Methods of Testing Blowing Fans. Prof. R. C. Carpenter. (47) March 24.
Some Notes on Oil and Tar Burning.* Benjamin J. Allen. (24) March 26.
The Present Status of Fuel Gas. John R. Lynn. (24) March 26.

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*Illustrated.

Mechanical—(Continued).

New Works of the William Krause & Sons Cement Company.* Horace De R. Haight. (14) March 31.
Limits to the Use of Forced Draft for Marine Boilers. Walter M. McFarland. (9) April.
Progress in Aerial Transportation.* William Hewitt. (10) April.
The Geometrical Generation of Irregular Surfaces in Machine Construction.* Henry Roland. (9) April.
Gas Engine Progress. C. V. Kerr. (41) April 1.
La Navigation Aérienne.* Rodolphe Soreau. (33) Serial beginning Feb. 24, ending March 10.
Transporteur Automobile sur voie Aérienne.* (35) March.
Chargement et Déchargement Mécaniques des Cornu à Gaz.* (33) March 10.

Mining.

The Transvaal Mining Industry for the Second Half of 1899.* (16) Serial beginning March 10, ending March 31.
The Avino Mine and Mill, Mexico.* (16) March 17.
The New Peck Montana Concentrator.* Geo. W. Winter. (16) March 31.
Flushing of Culm in Anthracite Coal Mines.* Wm. Griffith. (3) April.
Beach Sands of the Pacific Coast.* W. J. Adams. (41) April 1.
A New Method of Shaft Sinking.* G. C. McFarlane. (16) April 7.

Municipal.

The Action of Water on Asphalts. George C. Whipple and Daniel D. Jackson. (13) March 22.
Cement Macadam. (51) March 24.
Road Construction in New Jersey. (14) March 24.
New Standard Method of Testing Paving Brick; National Brick Manufacturers' Association. (13) March 29.
Smoothness of Pavements.* Daniel B. Luten. (14) March 31.
Failures in Asphalt Pavements and Their Causes. A. W. Dow. (13) April 5.

Railroad.

Locomotive Design.* F. J. Cole. (25) Serial beginning June, 1899, ending April, 1900.
Suggestions for Revision of the M. C. B. Rules for Loading Long Material. F. H. Stark. (61) Feb.
Ton-Mile Statistics. C. H. Quereau. (61) Feb.
Corrosion of Locomotive Boiler Tubes.* (50) March 1.
High Speed Three-Phase Railway from Toledo to Norwalk.* (17) March 3.
How to Determine the True Net Earning Power of Street Railway Properties. Edward E. Higgins. (17) March 3.
Main Power Station and Transmission System of the Metropolitan Street Railway Company of New York.* (17) March 3.
Prices of Street Railway Track Construction. John P. Brooks. (17) March 3.
Superstructure on the Vienna Stadt-bahn.* H. Koestler. (53) March 9.
Rails: Committee Report to the Annual Convention, American Railway Engineering and Maintenance of Way Association. (40) March 9.
The Erection of the Boston Elevated Railway.* Charles Evan Fowler. (13) March 15.
Ballasting: Committee Report to the Annual Convention, American Railway Engineering and Maintenance of Way Association. (40) March 16.
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Railroad Signaling and Interlocking Plants: Committee Report to the Annual Convention, American Railway Engineering and Maintenance of Way Association.* (40) March 16.
Railway Building in Prospect: Detailed List of New Railroads Projected and Under Construction. (40) March 16.
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Exécution des Tunnels de la ligne d'alacheir à Fium Karahissar (Asie Mineure).* (35) March.
Le Chemin de Fer Suspendu de Barmen-Elberfeld-Vohwinkel (Allemagne).* Alfred Boudon. (33) March 10.

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English Experiments on the Bacterial Treatment of Sewage, with an Account of the Work Done at Manchester, England, During the Past Year.* Prof. Leonard P. Kinnicut. (1) Feb.
Recent Changes in Sewage Disposal, Worcester, Mass. (14) March 17.
Tunneling in Quicksand.* H. P. Eddy. (14) March 24.
Garbage Reduction Works at Pittsburg and Allegheny, Pa. (13) March 29.
Distribution of Steam at Dartmouth College.* (14) March 31.
The International System of Bacterial Sewage Treatment. (14) April 7.
The Sewerage Problem of the City of Worcester, Massachusetts.* (19) April 7.

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Nickel Steel: Its Value as an Alloy. David H. Browne. (62) Serial beginning Jan. 11, ending Feb. 15.
Paints and Varnishes. Professor A. H. Sabin. (1) Feb.
The Testing of Struts or Pillars. W. C. Popplewell. (47) Serial beginning March 17, ending March 24.
Armored Concrete. (12) March 30.
A Great Arch, Cathedral of St. John the Divine.* (14) March 31.

Topographical.

A Method of Making a Farm Survey. G. B. Zahniser. (13) March 29.

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Calcul des Reservoirs en Tole.* Maurice Koechlin. (33) March 3.

Waterways.

Santa Ana Canal.* J. B. Lippincott. (19) Serial beginning March 10, ending March 17.
Difficult Repairs to the Alderney Breakwater.* (14) March 17.
Proposed Plan for Regulating Works for Controlling the Level of Lake Erie.* March 22.
To Raise the Level of Lake Erie. (20) March 22.
A Comparison of the Isthmian Canal Projects. George A. Burt. (9) April.
The Dutton Pneumatic Balance Locks for Canals.* Chauncey N. Dutton. (3) April.
Usine Hydraulique de Bourg-et-Comin pour l'Alimentation du Canal de l'Oise à l'Aisne. L. Lefort. (34) Serial beginning Oct., 1899, ending March, 1900.

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* Illustrated.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

THE CONSTRUCTOR.

A Hand-Book of Machine Design. By F. Reuleaux. Authorized Translation, complete and unabridged, from the Fourth Enlarged German Edition, by Henry Harrison Suplee, M. Am. Soc. M. E. Cloth, 12 x 9 ins., 312 pp., illus. New York, H. H. Suplee, 1899. $7.50.

The headings of chapters are: Strength of Materials; The Elements of Graphostatics; The Construction of Machine Elements; Riveting; Hooping; Keying; Bolts and Screws; Journals; Bearings; Supports for Bearings; Axles; Shafting; Couplings; Simple Levers; Cranks; Combined Levers; Connecting Rods; Cross Heads; Friction Wheels; Toothed Gearing; Ratchet Gearing; Tension Organs Considered as Machine Elements; Belting; Rope Transmission; Chain Transmission; Strap Brakes; Pressure Organs Considered as Machine Elements; Conductors for Pressure Organs; Reservoirs for Pressure Organs; Ratchets for Pressure Organs or Valves.

INDICATOR DIAGRAMS.

A Treatise on the Use of the Indicator and Its Application to the Steam Engine. By W. W. F. Pullen. Cloth, 9 x 6 ins., 238 pp., illus. The Scientific Publishing Co., Manchester, 1899. 6 shillings net.

In this volume, the author has endeavored to give the result of a certain amount of experience to assist in interpreting the record made by the indicator pencil, as well as to indicate in what direction to look for those irregularities commonly found under ordinary conditions of use. The Contents are: Measurement of Power with the Indicator; Indicator Rigs and Reducing Gears; Other Indicators; Calibrating Indicator Springs; Other Errors in the Indicator Diagram; Preliminary Analysis of the Diagram; Diagrams Showing Variation of Load; The Admission and Steam Lines of the Indicator Diagram; The Exhaust and Compression Lines; Valve Chest and Steam Pipe Diagrams; Adjustment of the Slide-Valve; Pump Diagrams; Miscellaneous Diagrams; Averaging Diagrams and Mechanical Efficiency; Slide Rule and Calculators; Tables.

MODERN LOCOMOTIVES.

Illustrations, Specifications and Details of Typical American and European Steam and Electric Locomotives. One-half Morocco, 15 x 11 ins., 405 pp., illus. The Railroad Gazette, New York, 1897. $7.00.

Most of the drawings in this volume have been furnished by the railroad companies or makers. The attempt has been made to limit the locomotives shown to those which are in present use, and to include fairly representative types of all steam and electric locomotives built in the United States. To these are added illustrations and descriptions of a number of foreign-built types. The selection of all the drawings, and the general arrangement of the volume, was made by the late D. L. Barnes, M. Am. Soc. C. E. After his death the work was completed by Mr. J. C. Whitridge.

THE FILTRATION OF PUBLIC WATER-SUPPLIES.

By Allen Hazen, Assoc. M. Am. Soc. C. E. Third Edition, Revised and Enlarged. Cloth, 9 x 6 ins., 321 pp., illus. New York, John Wiley & Sons, 1900. $3.00.

In the present volume the author has endeavored to explain briefly the nature of filtration and the conditions which, in half a century of European experience, have been found essential for successful practice, and of preventing the unfortunate and disappointing results which so easily proceed from the construction of inferior filters. In the five years since the first edition of this book was published, progress in the art of water purification has been rapid and substantial. A complete revision was required to treat these newly investigated subjects as fully as seemed necessary to the author. There is an index of five pages.

EGYPTIAN IRRIGATION.

By W. Willcocks, M. Inst. C. E.; with an Introduction by Major Hanbury Brown. Second Edition. Cloth, 10 x 7 ins., 485 pp., plates. London, E. & F. N. Spon, Ltd., 1899. $12.00. (Obtained by Purchase.)

This is a description of the irrigation works which have been built in Egypt in the last twenty years. The Contents are: Egypt; The Nile; Basin Irrigation in Upper Egypt; Perennial Irrigation in Upper Egypt; Perennial Irrigation in Lower Egypt; Egypt by Provinces; Drainage and Land Reclamation; The Barrages; The Nile in Flood; Engineering Details; Duty of Water and Agricultural; Administrative and Legal; Reservoirs.

ENGINEERING WORKS OF THE KISTNA DELTA.

A Description and Historical Account. Compiled for the Madras Government by George T. Walch, M. Inst. C. E. Cloth, 10 x 6 ins., 2 vols., 37 plates. Madras: Printed and Published by the Superintendent, Government Press, 1899. (Donated by F. J. E. Spring, M. Am. Soc. C. E.)

This is a history of the engineering works by which the Delta of the Kistna has been converted from a poverty-stricken, sparsely cultivated tract, subjected to recurring droughts, into a prosperous country, covered with cereal crops, rendered virtually independent of the precarious local rainfall by a network of canals and channels from the great river. The works include an anicut and head works and numerous canals.

NOTES ON THE NICARAGUA CANAL.

By Henry I. Sheldon. Second Edition. Cloth, 8 x 5 ins., illus., maps. Chicago, A. C. McClurg and Company, 1899. (Donated by the Author.)

This volume is the result of a visit to Nicaragua made by the author in 1895. The Contents are as follows: The Journey to Nicaragua; The Route of the Canal; Concessions and Legislation; The Present Plans; Sanitary Questions; The Country and the People; Other Great Canals; Cost; Nicaraguan Cities; Shall the United States Assist?

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Vol. XXVI. APRIL, 1900. No. 4.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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EXPERIMENTS ON THE PROTECTION OF STEEL AND ALUMINUM EXPOSED TO WATER.

Discussion.*

By Messrs. OSCAR LOWINSON and A. H. SABIN.

Mr. Lowinson.

OSCAR LOWINSON, Assoc. M. Am. Soc. C. E.—The speaker has made a number of tests of a new paint, and with a view to determining its resistance to moisture, and the effects of frost, a coat was applied to the outside surface of a porous jar which was afterward filled with a concentrated solution of sulphate of soda and the solution allowed to evaporate. After a period of two weeks the interior of the jar was full of crystals, and with the exception of half an inch from the top of the outside, which had been carried over by capillarity, the surface was still intact and showed no signs of having in any way been penetrated. This experiment was repeated with the same jar half a dozen times with the same results. A corresponding jar to which no paint was applied emptied in about 30 hours, both the inside and outside being covered with sharp crystals.

In order to determine whether this material could be used as a substitute for furring, a brick coated with it on its top surface was plastered directly on the paint and immersed to one-half its depth in a vessel containing water, until all the water, including that which the brick absorbed, had been evaporated. This process took a week, and showed absolutely no effect on the plaster. The plaster adhered so

*Continued from February, 1900, *Proceedings*. See October, 1899, *Proceedings*, for paper by A. H. Sabin, Assoc. M. Am. Soc. C. E., on this subject.

strongly to the paint that it could not be dislodged without destruction. Mr. Lowinson

A similar brick was plastered without paint, and in a week the plaster became loose and detached from the brick.

"Furring," which is used on the inside of exposed walls of buildings to prevent moisture from attacking the plaster and thereby disintegrating it or staining the finished wall surface, can be dispensed with by the use of this paint on the wall surface and the direct application of the plaster upon it. The speaker also uses it on the back of limestone set in masonry to prevent the discoloration of the wall, which occurs when any other cement than the expensive LeFarge is used.

The paint also appears to resist the attacks of moderate acids and alkalies, and herein differs apparently from the paints mentioned by the author.

Two coats were applied on a rusty, cast-iron, fire stand-pipe in front of a building in New York City where it has stood for a year without flaking or showing rust spots, and when last examined by the speaker it was still hard and adherent, and to all appearances had stopped oxidation.

The speaker has not been converted to the theory of the author that laboratory tests are not of much value. He firmly believes in exposure tests, but, in order to satisfy himself, deems it his duty, when examining a material like paint, to make such tests as he thinks would give an imitation of the action of the elements.

A. H. Sabin, Assoc. M. Am. Soc. C. E. (by letter).—None of the tests in this series was made with the samples alternately wet and dry. Some experiments, however, were made by the writer by coating bundles of wires, which were then attached to piles in the sea water in such a way that one end of each bundle was under water constantly, while the upper parts were exposed to the air at low tide. The conclusion reached from this and other experiments was, that the protective effect of the coatings was not lessened by the intermittent exposure, but that damage was done by floating objects in the water which persistently batter off the coating at the water-line. If secured against such effects protection is possible. This is probably the reason why iron piles rust off near the water-line; they are constantly battered by logs of wood and other floating matter which the waves throw against them incessantly. Mr. Sabin.

One of the objects sought by placing the plates horizontally in salt water was to get a growth of marine organisms on one side of the plate, while the other (the upper) side was kept free and reasonably clean. There was no considerable deposit of silt on these plates. In this way the exact effect of the organisms could be determined. The plates put in fresh water were laid in a vertical position because it was

Mr. Sabin. unavoidable under the circumstances, but as fresh water organisms are comparatively rare and did not attack the plates, it was no serious objection. The most serious trouble in these experiments was the injury to the coating on the edges of plates caused by floating objects in the water. This was sufficient, in some cases, to obscure the real value of the coatings, which were removed mechanically from the edges toward the middle of the plates. The writer is of the opinion that, hereafter, plates to be tested, which should not be in any case less than ⅛ in. in thickness, and should not be smaller than those used in this test—the larger the better—should each be put in a frame such as used for the slates of school children, in order to protect the edges as well as possible.

It is probably impossible to give any definite answer to Mr. Hill's question. The exposure in steel frame buildings is extremely variable. Good compact hydraulic cement mortar, unquestionably, affords a great deal of protection, because it is alkaline, and alkalies do not rust iron; and also because it prevents much circulation of air, though it does not do so absolutely. Ordinary lime mortar is so very porous that its value is much less; and brick and tile vary so much in porosity that no generalizations about them can be made. The care with which these are applied is also an important factor. It seems to the writer that the most important object to be sought in these cases is to keep the metal dry. It is not possible to keep a bridge or similar structure dry, but it is possible with the steel frame of a building; if it is dry it ought not to rust very fast. Foundations which are so placed as to be always damp should have their steel work bedded in cement, which should be of as perfect quality and as great thickness as possible.

The writer does not share in the prejudice which some have against painting metal which is to be covered with cement; it is not likely that the latter has any mysterious action on the iron, but acts as any other protective coating does; and it is well known that some paints which do not bear ordinary exposure last well under cement.

The writer has not used Lucol oil, and hence has nothing more substantial than a prejudice against it. He has preferred to keep to the use of materials of known composition, and which he thinks have yielded the best results up to the present time. It is well known, as has been pointed out by Mr. Tatnall, that oil-paint films are somewhat porous, and are often thrown off by corrosion underneath; and it has been the writer's aim to produce films which would not be open to that objection, but would protect the metal until the film itself is completely destroyed. The object of this paper is to show how this may be accomplished by the use of well-known materials, combined by well-known processes.

The writer cannot at all agree with the statement that the use of

varnishes is a matter of great uncertainty, but, in fact, believes the opposite, and desires by these tests to show what products are most suitable for use on engineering structures. Furniture made 150 years ago by the celebrated Martin, of Paris, is still distinguished by its beautiful varnish; while violins, ranging from 200 to 300 years old, are still covered with the varnish originally applied to them. Such material is worthy of systematic study. Mr. Sabin.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

THE FOUNDATIONS OF THE NEW CROTON DAM.

Discussion.*

By Messrs. E. SHERMAN GOULD, GEORGE W. RAFTER, L. J. LE CONTE and CHARLES S. GOWEN.

Mr. Gould. E. SHERMAN GOULD, M. Am. Soc. C. E.—Apart from its intrinsic interest, this paper possesses that of timeliness, in that its presentation coincides so nearly with the retirement of Mr. Fteley from the Chief Engineership of the Aqueduct Commission. It describes the progress of what we need not hesitate to style the most important engineering work of the day, from its commencement up to the date of its handing over, fully and successfully launched, by Mr. Fteley to his successor.

This very apt connection between the presentation of the paper and Mr. Fteley's retirement will probably strike all members of the Society. To the more limited circle, composed of members and ex-members of the Corps of Engineers of the Old Croton Aqueduct, the death of Julius W. Adams, Past-President Am. Soc. C. E., occurring so near the date of its presentation, lends to this paper an added appropriateness of time and place.

The preliminary studies for the original project, of which Mr. Benjamin S. Church was the author, namely, that of a dam at Quaker Bridge, and of which the present work will be the final out-

*This discussion (of the paper by Charles S. Gowen, M. Am. Soc. C. E., printed in *Proceedings* for January, 1900) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to May 25th, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

come, were commenced some twenty years ago under the direction of Mr. Gould. the late Isaac Newton, M. Am. Soc. C. E., then Chief Engineer of the Croton Aqueduct, by the late E. S. Chesborough, M. Am. Soc. C. E., and the late Colonel Adams. These studies mark the early dawn of the era of scientific high-masonry dam design in this country. The design and construction of what were previously considered high dams, of earth with a center wall of masonry, had already been brought to a high degree of perfection by Tracy, Campbell and Mr. George W. Birdsall, but a masonry dam, upwards of 100 ft. high, was essentially a new proposition. It was new and startling to many of us at that time to learn that the possible crushing of such a structure under its own weight alone, or under its own weight combined with hydrostatic pressure, was a factor of the problem most seriously to be reckoned with. It is probable that this point, and, indeed, the whole question of the profile of equal resistance of such structures, was first brought to the notice of the profession at large through a translation, made by the writer, by direction of Mr. Isaac Newton, of some chapters from Debauve's "Manuel del' Injenieur," of which a small edition was printed by the Department of Public Works for the use of its engineer corps, and copies of which found their way to a few other hands. Mention may also be made of two papers contributed about this time by the writer to *Van Nostrand's Engineering Magazine*. Later, the subject was fully developed by Edward Wegmann, M. Am. Soc. C. E., in his masterly and thoroughly exhaustive treatise on "High Masonry Dams."

In these studies, the type or concrete idea of the high masonry dam, which with true engineering instinct was seized upon and kept constantly in view, as a safe precedent, by the consulting engineers, was the dam across the Furens, at St. Etienne, France.

But all these particulars are now ancient history, and appeal only to the very limited number of original pioneers in the study of high dams. Their only general interest lies in the evidence which they may afford of the labor and research which characterized the earliest beginnings of the project now being carried out. It is doubtful if any engineering project in this country has ever been made the subject of so much laborious and painstaking study as that described. This paper, taken in connection with the reports of the Chief Engineer, already published, shows us that these studies, taken up by Mr. Fteley, where they were left off by the original projectors, were continued by him with unabated zeal and thoroughness.

The description of the system of borings and other explorations given by Mr. Gowen is noteworthy. The juxtaposition of gneiss and limestone, with outcrops on opposite sides of the valley, seems to be characteristic of this and the neighboring territory, and merits study by the geologist. The disappearance and in some cases reappearance

Mr. Gould. of water in the bore holes at great depths is certainly puzzling to account for. Mr. Gowen calls attention to the discrepancy frequently found to exist between the character of the rock as revealed by the actual excavations and that previously predicted from the borings. This discrepancy was also noticeable in driving the tunnels of the New Croton Aqueduct, and it admonishes us that while the diamond drill is of great utility in preliminary explorations, its indications should be taken with considerable reserve, and interpreted very cautiously.

The quotation from Mr. Fteley's Report (pages 16 and 17), in which he recommends abandoning the Old Quaker Bridge location and building a much smaller dam higher up the stream, is an excellent example of sound engineering judgment, and one is rather surprised at the haste with which, in the face of this recommendation, the Commission adopted the Cornell project. The fourth reason advanced by Mr. Fteley for his recommendation seems, however, to require some modification. He says:

"The interest of the money thus saved for the present would, after twenty-five years, represent a large part of the money necessary to then build the higher dam, with the result that the city would then have two dams instead of one for nearly the same expenditure."

This result could only be safely predicated if the amount of interest saved were year by year paid into a sinking fund, and kept intact. This, it is hardly necessary to remark, would be very unlikely to be carried out.

In this connection, a glance at the estimated amount of storage, consequent upon the completion of the present dam, will be interesting. On page 4 this amount is stated to be 73 236 000 000 galls. The writer has found that a very serviceable formula representing the relation between the total storage required to maintain a desired daily average consumption throughout the year, and the daily average yield of the source of supply, is:

$$S = \frac{C^2}{Y} \times 365.$$

In this equation, $S =$ total storage required; $C =$ daily consumption, and $Y =$ daily average yield of the water-shed, all in the same unit. Let us apply this formula to the present case. The capacity of the New Croton Aqueduct is about 300 000 000 galls. per 24 hours, and this may be taken as the maximum daily consumption of New York, furnishable by this supply. The daily average yield of the Croton watershed above the Cornell Dam, will be about 365 000 000 galls. per 24 hours, We would have then:

$$S = \frac{90\ 000}{365} \times 365.$$

$$S = 90\ 000\ 000\ 000 \text{ galls.}$$

This would be about 300 days' supply, as against 244 actually provided, indicating a fair agreement between the two figures. Mr. Gould.

Although this paper is confined to the foundations of the great dam, it cannot be satisfactorily discussed without some reference to the profile of the dam itself, which is shown in Fig. 2.

The feature of the profile which immediately challenges discussion is its form below the level of the original surface of the ground. It will be perceived that the profile of equal, or approximately equal, resistance is continued down to the bed rock, some 120 ft. below the bed of the stream. The effect of this is to spread greatly the footing course, and to increase correspondingly the immense volume of material to be excavated. Was this necessary, to insure the required stability of the structure? The writer has no hesitation in saying that both from theoretical and practical considerations he does not believe it to have been necessary or even advisable. He understands that the theory upon which this profile is based is that the dam is, or may be, subjected to the pressure due to a head of water extending from its top, through the excavation and down to the bottom rock on which it stands. He considers this theory as inadmissible. Its acceptance appears to lead to the untenable conclusion that the deeper the foundation, the greater the hydrostatic stress. We have only to consider what a monstrosity would ensue if the dam were carried down to a very great depth, say 300 ft., below the surface and the profile calculated according to this theory. He considers also, that this endeavor to err, if at all, on the side of safety is, to some extent, self destructive, for it results in piling up a huge mass of back-filling upon the outer toe of the dam, adding this extra weight to the already enormous pressure which it is sustaining. The additional spreading of the foot cannot be necessary to resist rotation about the outer toe, because such rotation would be impossible at that depth, nor can it be needed to prevent crushing. That part of the structure which is deeply buried and closely imprisoned on all sides is quite differently and much more favorably circumstanced to resist crushing than is that part which stands entirely above ground with no opposing resistance to prevent the lateral escape of crushed material. In this connection, the writer would quote what he has said elsewhere, as follows:

"Nor can he agree with those who maintain that the thrust of the water from a full reservoir should be considered as that due to a head extending from the top of the dam to the bottom of the foundations. That portion of the dam which is buried in the earth or rock should, in his opinion, be considered entirely apart from the dam proper, and as subject to an entirely different class of stress. He would consider this portion of the structure as forming, in fact, a part of the geology of the territory, and confine his calculations, as regards the thrust of the water, to the superstructure which, standing in relief above the surface of the surrounding ground receives the pressure of the water on one side, and that of the atmosphere only on the other."

Mr. Gould. Apart from this feature of the profile, some remarks may be made upon the dimensions given to the upper portion. The calculations which lead to the profile adopted were based upon the well-known empirical formulas for resistance to crushing, sometimes called Debauve's formulas, from the fact that they are given by that author in his "Manuel del' Injenieur," though not original with him. It is usual to assume that if the profile, so calculated, satisfies the condition of resistance to crushing, it will of necessity satisfy also that of resistance to overturning. This method of calculation results, as regards the latter condition, in a profile which is very light in the upper portion and very heavy in the lower.

In looking at the profile of a very high masonry dam on paper the eye deceives us, unless we keep the scale in mind. We are accustomed to regard such structures—dams and other retaining walls,—in reference to their apparent stability as against overturning bodily around the outer edge. Regarded in this way, the profile always seems to be, and indeed is, skimped at the top and redundantly thick at the bottom. This appearance changes, however, when we reflect upon the character of the stress brought to bear upon the base and all the lower portions of the wall. The tendency of this stress is to pulverize the bottom courses, and it can only be resisted by an extension of base beyond what would be required to secure a sufficient moment of resistance to overthrow. To use a homely figure, the dam may be compared to a sack of meal in danger of bursting, rather than to a rigid body in danger of toppling over.

In designing very high dams, therefore, foreseeing as we do the width of base to which we will be forced in order to meet the increasing crushing stress, we instinctively economize in the upper portions, influenced partly, no doubt, by the fact that if failure is to occur anywhere, it had better be at the top than the bottom, and also to make the unit stresses more uniform throughout. But this economy or this desire to equalize crushing stress may be pushed too far. It seems to the writer that this has been done in the present case, where, in his judgment, the outer face of the dam has been hollowed out too much above Elevation 100.

It is stated in the paper that the dam has a general factor of safety of 2 against rotation. It is understood that in all calculations the water in the reservoir is supposed to stand level with the top of the dam, or at Elevation 210. This is certainly an extreme assumption. The spillway elevation is 196, its length 1 000 ft., and its required capacity rated at 15 000 cu. ft. per second. This would correspond to an elevation of about 199, or say 200 at most, in the reservoir. It is probable, therefore, that the dam above Elevation 100 has a factor of safety of at least 2.5, and a rough calculation seems to show this to be the case. But it must be borne in mind that these calculations

assume a purely static pressure, due to absolutely quiescent water. Mr. Gould. This dam, however, will act as the retaining wall, or breakwater, of an immense and deep lake, with soundings of 140 ft. in direct contact with an almost vertical back. Over this lake violent storms will rage, accompanied by wave action of tremendous dynamic force. Is this part of the dam sufficiently massive to meet the shock of these waves and hurl them back upon themselves? Hugh fields of floating ice may be expected to thump heavily against the wall; is it heavy enough to resist their impact urged on by wind and waves? To say the least, we must admit that the practical factor of safety is reduced to its absolute minimum.

The writer would be in favor of placing an earthen embankment, well rip-rapped at the back of the dam, for a portion of its height. The effects of such a bank would be to diminish the chance of percolation down the back and reduce still further the bugbear of an exaggerated hydrostatic pressure; to diminish considerably the effect of the deep wave action against the dam, and by maintaining a constant counter pressure against the back, to limit the range of pressure when the reservoir is alternately full and empty. It would be especially advisable to cover the entire area of the refilled excavation within the reservoir with a heavy embankment in order to consolidate by its pressure the material used in refilling. Plate I shows that the inner slope of the earthen dam on the south side already covers, or is to cover, a large portion of this refilling, so that only an extension of the bank is necessary in order to carry out the suggestion.

From the start, it was recognized that the chief engineering difficulties to be overcome in the building of the great dam were the diversion of the river and the taking out the foundation pit. There can be no doubt that the plan pursued to divert the stream was the best, and its complete success, as recorded in the paper, is the just reward of an intelligent design skillfully carried out. There is no doubt, too, that the method adopted for taking out the excavation, by means of an open cut with sloping sides, was the best under the circumstances, and more certain of success than any attempt to shore up the sides could have been. Had the superstructure been designed to rest upon a base with vertical or nearly vertical sides, the suggestion made originally might have been revived; namely, to take out two comparatively narrow trenches, one at the up-stream and the other at the down-stream face of the foundation, sustaining the sides by means of shoring, and building up these two faces first. The central core of earth could then be removed between these two walls, and the remaining masonry laid. Even in this case, however, the surer plan of side slopes might have been found preferable. Be that as it may, the work has been accomplished successfully as described, and at the present time, when the critical period has been safely

Mr. Gould. passed and the foundations brought up to surface level, there can be nothing but congratulations to all concerned in carrying through this bold and brilliant feat of engineering to a triumphant issue.

The successful prosecution of the work below ground depended upon the ability to keep the pits dry. Evidently this fact was realized fully by the engineers, and a powerful pumping plant installed for the purpose It is not always thus, and many operations involving deep excavations are increased greatly in cost, difficulty and danger by inadequate and badly managed pumping facilities.

The overflow arrangement seems to have been intelligently planned, and from an examination of the plans and descriptions contained in this paper the writer thinks it would be difficult to find a flaw in the general design of the work, as regards the handling of the water before, during and after construction.

An interesting feature of the work is the earthen dam with core wall on the south or limestone side of the stream. The writer has not noticed any explicit statement in the paper as to why the change was made from masonry to earth, but it may be inferred readily that the limestone rock was not considered sufficiently solid to warrant a masonry dam, pure and simple, for the entire length. If this was the case, then sound engineering judgment was shown in changing over to earth. It may be suggested, however, that the masonry dam might have been continued across the valley, leaving its down-stream face exposed, while the back was protected by the earthen bank. This would have involved more masonry, the cost of which would be partially offset by saving the outer wing-wall and outer earthen embankment.

In describing the masonry core-wall introduced in the earthen bank, Mr. Gowen speaks somewhat apologetically of its massiveness. In the writer's opinion it errs in the other direction, and should have been considerably thicker than shown. Its top should, by all means, be carried to Elevation 210, the same as the crest of the masonry dam. Carrying the top of the earthen embankment to 220, as shown in the drawings, is excellent judgment.

In the concluding paragraphs of the section describing the protective work (page 20), Mr. Gowen also speaks somewhat apologetically of the great cost of this work. No word of apology is necessary to justify this entirely wise expenditure. Parsimony here would have been the falsest economy.

The account of the manner of carrying on the deep excavations and of dealing with springs and fissures of the rock is very valuable. It appears to have been successful, and in any event is not open to criticism. This work could only be judged on the ground, and while actually going on. At present it is sufficient to know that what was attempted was accomplished successfully.

Something is said on pages 29 and 30 about a possible upward

water pressure against the bottom of the foundations of the dam. Mr. Gould. The writer considers that all apprehension of danger from such action is groundless. When the small proportionate area which in any event could be exposed to this action is taken into consideration; when capillarity and friction are given their due weight, and when it is remembered that at the worst it would be a hydraulic, not a hydrostatic, pressure that could take effect, as there would always be a line of escape below the dam, it will be realized that this danger dwindles down to a negligible value. The writer would class fears of this nature with those which prompt a continuation of the theoretical profile down to the deep-seated rock.

All the foregoing remarks apply to what may be called, distinctively, the engineering features of the work. The constructional details are also very interesting.

Before commencing to lay the masonry in the foundation pit of the main dam, it is stated that the rock bottom was painted with a grout of neat Portland cement, which was allowed to set before commencing to build. The writer would question the wisdom of interposing a film of cement between the rock and the masonry. He would prefer to bed the stones directly upon the sharp, clean rock.

The arrangement of the derricks and the racking of the work as described and shown on Plate X, Fig. 2, were judiciously planned, and calculated to secure rapid, systematic and substantial work. The cable-way does not seem to figure in this part of the work. It is not stated whether the "Portland" mentioned was American or foreign, nor whether the "American" cement was of the Portland or Rosendale type. This leaves us a little in the dark respecting the relative merits of the American mortar, 2 to 1, and the Portland, 3 to 1.

The stone used under the name of "gabro" is probably a syenite, differing from granite or gneiss in that the quartz is replaced by hornblende.

The precautions taken in bedding the stones, described on page 58, are those necessary to secure good hydraulic masonry. It is probable, however, that as the work progressed and the gangs became broken in to the requirements of the inspection, the beds were properly prepared at once, without the necessity of raising every stone in order to rebed it. Foremen accustomed only to ordinary first-class masonry, such as bridge abutments, are apt to be dismayed at first by the lavish use of mortar required in hydraulic work. They soon realize however, that, even when mortar is thrown in by the shovelful none need be wasted, for the surplus is forced out by the stones as they are laid, and goes to form the bedding of the neighboring ones. Specimens of the work are shown in Plate XII, and to a larger scale in Fig. 2, Plate XI. In the latter figure some hammer-dressing or possibly "plug and feathering" seems to have been used to secure

Mr. Gould. approximately vertical joints and horizontal beds, and in these respects the work is satisfactory. It must be recognized, however, that the stones are badly shaped for substantial work. Unless they are all headers, which would be bad construction, they are nearly all too high in the rise for the length of bed, making them top heavy and rendering it next to impossible to secure a good bond, as is plainly seen in the figure. In a wall of the immense thickness of the main dam it is true that many defects of this sort are of comparatively little moment, for the wall must be considered in the mass, and besides, true Portland cement mortar proportioned 2 to 1 or even 3 to 1 is a tower of strength, and covers many sins. But in the case of light work, such as the center wall of the earth embankment, the stones should be got out in such shapes that each individual piece is in stable equilibrium when laid in the wall with its best bed down. They should admit of being laid readily, with a perfect interlocking bond, every vertical joint being well capped by the stone above it, the bond being maintained, not only on the face, but throughout the entire body of the work. In Fig. 1, Plate XII, showing the unfinished end of the spillway, a tendency may be seen to produce a triple wall, that is, to build the two faces first, and fill in between them afterward. Unless the greatest care be taken to prevent it, thin work will almost always be built this way, but the result is a weak combination, and the tendency should be carefully guarded against.

The prices paid for the different classes of work furnish valuable economic data. The price of rubble seems very low, although the facilities for using large stones and working generally to good advantage, are very great. The prices paid for excavation, both rock and earth, on the other hand, seem high. No price is stated for refilling or embankment. It is noteworthy that no concrete is mentioned.

The over-running of the time limit in a work so carefully planned, and in which there were no blunders to correct, nor extensive additions made after the work was commenced, is suggestive at the present time when other gigantic undertakings are contemplated by the city.

Mr. Rafter. George W. Rafter, M. Am. Soc. C. E. (by letter).—The portions of this paper relating to the borings for determining the nature of the foundations are especially interesting to the writer, as well as the observation that, in the Croton Valley, wherever the bed-rock appeared at one side, it almost invariably dipped down sharply on the other side to a depth at which it would be impracticable to establish a foundation.

This general condition, or a modification of it, is found frequently in streams issuing from the granitic rock horizons of the Adirondack mountains of New York. The writer has found it repeatedly in his extended series of examinations of sites for storage dams in that region.

As to why this phenomenon repeatedly occurs, the geologists are silent; and thus far the writer has been unable to assign any explanation. Certainly, in view of the large amount of high dam construction now projected in the State of New York, an answer to this question would be useful, if for no other purpose than to tell us in many cases what to avoid. An answer is desirable further on the general principle that, with the reason fairly understood, we may hope to find more easily the point of minimum resistance—that is to say, the location on a given stream where the conditions are, on the whole, the most favorable. Mr. Rafter.

As to a solution of this problem, the writer considers that undoubtedly it will come through a better understanding of the laws governing glacial drift and the complex phenomena of surface geology, generally.

As stated, some of the Adirondack streams present a modification of the condition described by Mr. Gowen, namely, the bed-rock frequently shows on one side of the valley, dipping down to a few feet below the bed of the stream and then running off on the other side either horizontally or approximately so. This was the condition at Indian Lake, where a dam 47 ft. high was erected in 1898. So far as the studies have been carried, it is the condition at Boreas and Cheney Ponds, Tumblehead Falls, Conklinville and other points proposed as sights for high dams in the Adirondack region, and of which some of the details may be obtained by reference to the writer's reports on the Upper Hudson storage surveys.

In 1893–96, the writer made a series of studies for high dams on the Genesee River ranging from about 100 ft. to 175 ft. in total height. In the work in the Genesee Valley, in 1893, the rocks dealt with were the rather soft and friable Genesee shales. In general terms, the problem was to find material hard enough to carry securely the superimposed weight and, at the same time, insure water-tightness under the foundations and at the ends.

It was deemed desirable to use the diamond drill extensively. The cores taken out showed that the first 20 to 30 ft. of the rock foundation, while evidently capable of carrying the proposed loads, was defective in that there were many minute seams through which considerable water might be expected to escape when under pressure. In order to gain some idea of just how serious a matter this might be, water pressure was applied to drill holes after the drilling was finished, by the use of a rubber packer, placed at different elevations in the holes. Water was forced below the same by means of a pipe passing through the packer. Space will not be taken to describe in detail the arrangements for accomplishing this, because illustrations of such rubber packers may be found in the catalogues of firms dealing in well supplies. Moreover, the appliances and the results attained

Mr. Rafter. have been described by the writer somewhat in detail in his reports on the Genesee River storage, for 1893 and 1894. The object in referring to the matter at all on this occasion is chiefly to complete the literature of the subject in the *Transactions* of this Society. So far as the writer is aware, his methods of using water under pressure for testing the quality of rock foundations, as worked out in 1893, were somewhat in advance of methods used previously.*

To illustrate the methods used and the results obtained in 1893, the following abstracts from the log of the tests, as kept from day to day, are presented:

October 17th, 1893.—Tested drill hole *YY4*, using rubber packer set 50 ft. from top of casing. On starting pump, gauge showed 60 to 70 lbs., and pressure rose gradually to 110 lbs. At this pressure the hole took all the water the pump could deliver. After pumping for an hour with the packer at Elevation 539.5, disconnected and found that water ran slowly from top of pipe, about 5 ft. above surface of ground (Elevation 594.0), thereby showing that a small head had been gained at the sides. Packer was then raised to Elevation 559.5 (top of rock at 565.2), and the pump again connected. With the pump at full capacity, the pressure was only 20 lbs. and no more could be gained however rapidly the pump was run. The clear inference is that between Elevations 539.5 and 559.5 there are seams or fissures which allow water to flow out of the drill hole when under about 20 lbs. pressure.

October 19th, 1893.—Tested drill hole *ZZ1*. Packer was first set 19 ft. above bottom (Elevation 532.5). Pump stalled at 100 lbs. pressure. Raised packer gradually, packing it at every few feet by setting pressure above against pressure of water from below. In this way the hole was tested for its entire length and found to stand, as stated, 100 lbs. at the bottom, and from 40 to 50 lbs. in the upper part.

October 31st, 1893.—Tested horizontal hole at foot of Hog-back. Set packer 87 ft. in, or 8 ft. from end. On starting pump, gauge showed 100 lbs. and rose gradually to 140 lbs. Released packer and stopped every 10 ft. until 54.5 ft. from bottom was reached. At this point gauge dropped to 100 lbs., but in 30 minutes advanced again to 140 lbs., and in 15 minutes more to 150 lbs., where it remained for 30 minutes and then dropped to 60 lbs., the steam pressure remaining the same. In 1 hour and 45 minutes the pressure was 40 lbs. At this point water dripped from the side of the Hogback for some distance to the South.

November 2d, 1893.—Repeated the foregoing test with packer 55 ft. from bottom of hole, and with 4 qts. of wheat bran below packer. Pumped with 100 lbs. pressure for 2 hours without effect.

November 2d, 1893.—Tested hole 23 + 42, W. 250, at Hogback location. Hole 84 ft. deep, 14 ft. to rock (elevation of bottom 503.4, top of rock 571.4). Set packer 8.5 ft. from bottom. Gauge showed 165 lbs. Raised packer to Elevation 524.4, and pressure dropped to 80 lbs. Disconnected and put in 4 qts. of bran, whereupon pressure rose to 170 lbs. Raised packer to Elevation 534.0 and still maintained same pressure. At Elevation 536.0 pressure dropped to 100 lbs. and

* Report on Genesee River Storage Surveys, Annual Report of the State Engineer and Surveyor of New York, for 1893, p. 416. Also, same report, 1894, p. 360.

remained at that point even after the addition of 5 qts. of bran. Mr. Rafter. Pressure finally fell to 60 lbs. and remained there for 3¼ hours. While pumping this hole, water ran from casing at 23 + 42, W. 350.

November 4th, 1893.—Tested 23 + 42, W. 350, at Hogback location. Set packer 10 ft. from bottom (elevation 518.0), and obtained 40 lbs. pressure. Added 4 qts. of bran without effect on the pressure. In 2¼ hours the pressure rose gradually to 65 lbs. Coloring matter was added, and showed in the water flowing from casing at hole 23 + 42, W. 250. On stopping pump, it was found that water pumped into hole had acquired a back pressure of 20 lbs. On disconnecting, water ran from pipe for 1 hour and 49 minutes. This test indicates not only a connection between hole 23 + 42, W. 250, and this one, under the river bed, and independent of it; but also shows backing up, probably in vertical seams, at the sides of the gorge. A number of other tests at this site gave the same result. In one of them the back pressure increased gradually to from 50 to 60 lbs., where it remained stationary during 4 hours' continuous pumping. Water was then discovered running from a fissure in the rock side of the gorge over 100 ft. above the river surface and several hundred feet away.

November 24th, 1893.—Tested B. 40 + 70, W. 750, at Site No. 1. Set packer 6 ft. from bottom. Pressure rose to 180 lbs., when pump stalled. Raised packer 10 ft., or to 16 ft. from bottom, when pressure rose at first to 170 lbs., but in a few minutes fell to 120 lbs., where it remained for 10 minutes, and finally fell to 100 lbs. Uncoupled and added bran, when pressure rose from 100 to 120 lbs. Raised packer to 26 ft. from bottom and gauge showed 40 lbs. Again added bran and gauge rose to 50 lbs. With packer at this elevation gas issued from casing at hole B. 40 + 70, W. 850. Upon raising packer 2 ft. more, larger quantities of gas flowed from the hole. The packer was raised and lowered several times with like results, showing a connection between the two holes at about Elevation 548.0.

The use of wheat bran, as referred to in the foregoing, was for the purpose of determining whether or not the seams permitting the escape of water were of minute or open texture. When minute, the fact was shown quickly and easily by the use of a very small quantity of bran.

In 1896 the final studies for the Genesee storage dam at Portage were made. Here very simple conditions prevailed. The rocks dealt with were the comparatively hard sandstones of the Portage group, and a few borings followed by water-pressure tests removed all uncertainty as to the nature of the foundation. The writer has no doubt that diamond-drill cores and a series of water pressure tests properly carried out can be made to yield more for a given expenditure than any other method of investigating this specific problem thus far devised. Indeed, there is really no other satisfactory method of investigation.

In view of the interesting and valuable results obtained, even from observation of loss of drill water at the New Croton Dam, it seems unnecessary to dilate at length on the practical value of such tests.

The method of stopping cavities by forcing in plastic clay is interesting and undoubtedly new to most engineers. Mr. Gowen is fortu-

Mr. Rafter. nate in the considerable number of either new, or substantially new, details developed on the work under his charge, and which he has presented in this paper.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper is a valuable contribution to practical knowledge on masonry dam building, particularly where the bed-rock at the site is of inferior character. It contains many interesting data relating to the treacherous nature of limestone formations. Every student of the stability of masonry dams, however, will feel a certain amount of disappointment in the fact that systematic efforts were not made to determine the amount and extent of the expected up-lift on the base of the dam, due to the upward pressure of the ground-water when the lake is filled.

It will be remembered that during the building of the Vyrnwy Dam Mr. Deacon went to much expense and made many useful experiments with the view of determining the amount and extent of up-lift on the base of the dam, and the results were both instructive and thoroughly convincing.

The mammoth dam now being built on the Cornell site, based on a seamy bed-rock full of running water, certainly furnished rare opportunities for making further valuable experiments in the same direction, and it seems strange that some efforts were not made to get more extended information on this all-important subject, and at no great additional expense. It is extremely doubtful how much of this up-lift can be suppressed by grouting wet seams and filling up cavities in the limestone bed-rock. Where hydrostatic pressures are great, as they will be in this case, it is hard to understand how the up-lift will be confined to the mouths of the bed-rock fissures exclusively.

The author states that a great many test holes were drilled and piped, the ground-water in some rising 83 ft., with the lake as yet empty.

It is to be hoped that some of these pipes have been, and will be, maintained and continued up vertically through the completed dam, with the view of noting the changes in the pipe water-levels as the lake fills up.

Mr. Gowen. Charles S. Gowen, M. Am. Soc. C. E. (by letter).—In presenting his paper on "The Foundations of the New Croton Dam," the writer did not anticipate that the question of the section adopted would be raised as the principal point of one of the discussions offered, and he would, for obvious reasons, prefer to leave the further discussion of this point to those who were originally more interested than he in the matter. Nevertheless, the following is offered in reference to this question, in connection with his reply to the other points raised by Mr. Gould.

As the writer understands it, the following were the principal conditions governing the design of the section:

The water level when the basin was full was taken at Elevation 206.

Water pressure was assumed to obtain to the level of the bed-rock surface. Mr. Gowen.

The back pressure due to the water on the down-stream side (water-table level) was also taken into account and allowed for.

Pressures (calculated) were limited to 15 tons per square foot at the base of the structure (rock surface), and the lines of pressure were kept well within the middle third of the section at any assumed level.

The above conditions are stated because Mr. Gould seems to have been in error in understanding that, in the calculations, Elevation 210 was assumed as high-water mark and that the profile of equal resistance was continued down to bed-rock, 120 ft. (instead of 75 ft.) below the bed of the stream; while, as to the elevation of the overflow (196), provision has been made for an increased height by means of flash-boards at some future time, and a high-water elevation of 206 is not improbable.

No additional thickness of section was made on account of possible ice pressure or wave motion. It is fair to assume that the extensive overflow located in close proximity to the main dam will operate at ordinary high water to relieve the lake of extensive areas of ice nearly as effectively as if it were on the main dam proper, while the width of the structure at the top (Elevation 210), necessary for a roadway, gives additional weight and stability to the section immediately below.

Mr. Gould maintains, apparently, that no account need be taken of pressures below the restored natural surface, below which the foundations may lie, and that no spreading of the "foot" should take place below this level. In other words, that the structure should be designed only with reference to its height above the restored natural surface; that the base on which it rests should be limited by vertical sides; and that dependence should be placed upon the weight of the refill for resistance to crushing and overturning, both of which tendencies would necessarily be increased greatly by the narrow foundation width. In support of this assumption he has stated that "he would consider this portion of the structure as forming, in fact, a part of the geology of the territory," which would seem to mean that the foundation wall must be taken as equal to the ledge rock below, at least in its capacity to resist crushing strains, and that the refilling or restored material must be as compact as it was originally, before excavation was made, in order to be as effective as possible against tendency to overturn. Is not this assumption extreme?

Further on, Mr. Gould states that the tendency of the stress, on the base and lower portions of the wall of a high masonry dam, is to pulverize the bottom courses, and that it can only be resisted by an extension of the base beyond what would be required to secure a sufficient moment of resistance to overthrow. He then compares such a section, perhaps not inaptly, to a bag of meal in danger of bursting. When,

Mr. Gowen. however, he proposes to extend such a section for an indefinite distance below the ground surface, and depends upon the refilling to counteract the bursting and overturning tendencies, it would seem to the writer that the plan is somewhat analogous to that of packing the bag of meal in shavings to avoid further chance of rupture to the bag.

It may be of interest to compare results obtained by Mr. Gould's formula for determining the required storage, with results derived by other methods, and if in case of the New York City water supply we assume $C = 280\,000\,000$ galls. per day—which is all that the Croton is calculated to supply in dry years—and $Y = 360\,000\,000$ galls. per day, which may be taken as a fairly conservative estimate of the average yield of the water-shed, as deduced from observations extending from 1870 to 1894, inclusive, we have as the required storage—

$$S = \frac{280\,000\,000^2}{360\,000\,000} \times 365 = 79\,490\,000\,000 \pm \text{galls.}$$

Deductions made on the basis of the Sudbury River records from 1875 to 1895,* inclusive, would give the following as the required storage for a dry year supply of 280 000 000 galls. per day—

Area of Croton water-shed 361 sq. miles.

$$\frac{280\,000\,000}{361} = 776\,000 \pm \text{galls. per square mile average yield}$$

per day required.

This requires a storage per square mile of water-shed of 200 000 000 galls. to prevent deficiency in a dry year, or a total storage of $200\,000\,000 \times 361 = 72\,200\,000\,000$ galls.

Later computations of the yield of the Croton, in which the gaugings of flow have been continued nearly up to date, show a continued close approximation between the actual average daily yield and the required storage as deduced through the medium of Mr. Stearns' tables based on the Sudbury flow, and the actual storage planned.

The question of trenching the outline of the foundation in order to save the excavation of self-supporting slopes was fully considered, in connection with the general problem presented in the matter of the earth excavation at the dam, and the writer is of the opinion, judging from the experience had with this part of the work, that neither time nor expense could have been saved by such methods. The maintenance and drainage of trenches sufficient for the purpose, which on the limestone side of the foundation would have had to be carried to varying and great depths into the bed-rock, would have proved especially expensive, as well as tedious, and, even with the trenches established and the retaining walls built in them, the resumption of the interior excavation work would have tended to cause delay and confusion, as the excavation and masonry work would have had to be

* "Suggestions as to the Selection of Sources of Water Supply," by F. P. Stearns, M. Am. Soc. C. E. Report of Massachusetts State Board of Health, 1890.

carried on in close proximity in a place where lack of working room was always found to be a hard condition to meet. Mr. Gowen.

The main consideration governing the change resulting in extending the main dam farther into the side hill was the reduction of the height of the embankment at the south end of the dam. The maximum height of the embankment above the level of the restored surface will now be about 50 ft. Another advantage gained is that the hardpan of the core-wall trench extends to the rock foundation for the full length of the trench, up to the juncture of the wall and the main dam.

The extensive bank of hardpan on the south side of the valley made it practicable to introduce the core-wall and embankment features into the design of the dam and was, in fact, one great reason for seriously considering this location as feasible and advisable, under certain circumstances, in the beginning.

The core-wall is planned to stop at Elevation 200, as the water in the basin will not rise above this level except for short intervals of time, and the embankment, at this elevation, will be fully 180 ft. thick and well paved.

As to the possible upward pressure due to percolation under the foundations: Mr. Gould alludes to fears on this score as groundless. In this the writer fully agrees with him, but it must not be lost sight of that Mr. Deacon, in building the Vyrnwy Dam, established a series of collecting and discharging drains in the body of the dam, in anticipation of possible percolation tending to influence the structure's stability.

The query in regard to the cement used is pertinent, and a more definite statement is warranted. The Portland cement thus far used is American Portland, Giant Brand, while the term "American cement" alludes to light-burned cement. Of this, large quantities have been used—mostly of the Union and Bridge Brands—the former a Lehigh Valley and the latter a Rosendale cement.

Regarding Mr. Gould's reference to Fig. 2, Plate XII, and his criticism of the shape of the rubble stones used for the up-stream facing, it may be said that in the effort to build a face with as small an amount of joint surface (requiring spawling and pointing) as possible, the ordinary rules governing the proportions between the height and width of stones may have been ignored at times. This, it would seem, was warranted when the great thickness and monolithic character of the structure is considered.

The writer regrets that Fig. 1, Plate XII, should show a questionable streak near the middle of the unfinished end of the spillway. This streak was due to the wash of surface material from above, and he is glad to assure Mr. Gould that the workmanship at this point compares very favorably with that at any other point of the structure.

Mr. Gowen.

Mr. Rafter's account of the use of a rubber packer in connection with the diamond drill is interesting and valuable, and the writer fully agrees with him that through the water pressure tests opportunity is offered for very complete utilization of diamond drill borings.

The results shown in the quotations from the log of the pumping tests are remarkably well defined, and the back pressures obtained and flows traced, at a distance and at comparatively high elevations, are notable, if only as indicating the extent to which such tests can be used in the examination and tracing of particular seams, as well as masses of rock in general.

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PAPERS AND DISCUSSIONS.

THE IMPROVEMENT OF A PORTION OF THE JORDAN LEVEL OF THE ERIE CANAL.

Discussion.*

By Messrs. EDWARD P. NORTH, JAMES OWEN, GEORGE HILL, J. G. TAIT, SAMUEL WHINERY, L. J. LE CONTE and CLIFFORD RICHARDSON.

EDWARD P. NORTH, M. Am. Soc. C. E.—Mr. Rafter's remarks are pertinent, but possibly he has missed, or has not emphasized sufficiently, the principal difficulty, namely, the errors made in planning the work. These errors may possibly be shown to the best advantage by Table No. 4, which gives the Engineer's estimate of quantities and prices, the contractor's bid on those quantities, and the approximate final estimate as given by the author. Mr. North.

It was estimated that more than 6 miles of swamp could be drained and kept free from water at a cost of $1 500. It was this lack of appreciation of the influence of water on marl and clay that nullified the contract; for, after the first season, there was virtually no contract between the State and the contractor, but, to use the words of L. E. Cooley, M. Am. Soc. C. E., "the contractor had a license to prosecute the work at his own price and on his own specifications."

It was well known that with the first canal, built in 1816 to 1825, there was much difficulty on what became known as "the Jordan Level." It was not then as notorious as it was under the enlargement of 1836. The author has stated that three contractors abandoned the work, and that, eventually, the bulk of it was done by the State. The

* Continued from February, 1900, *Proceedings*. See December, 1899, *Proceedings*, for Paper by William B. Landreth, M. Am. Soc. C. E., on this subject.

Mr. North. TABLE No. 4.—Preliminary Estimate, Successful Bid and Approxi-

Engineer's Estimate.

Quantities.	Units.	Items.	Prices.	Aggregate.
1	....	Grubbing and clearing	$500.00	$500.00
1	....	Bailing and draining	1 500.00	1 500.00
150 000	Cu. yds..	Dry excavation of earth	0.27	40 500.00
4 500	" ..	" " rock	1.00	4 500.00
500	" ..	Excavation of masonry	1.00	500.00
5 000	Sq. ft....	Rock channeling	0.25	1 250.00
1 000	Cu. yds..	Embankment	0.25	250.00
17 000	" ..	Lining	0.60	10 200.00
1 000	" ..	Puddling	0.15	150.00
500	Lin. ft...	Piles delivered	0.15	75.00
500	" ..	" driven	0.10	50.00
50 000	" ..	Piles at foot of walls delivered	0.10	5 000.00
45 000	" ..	" " driven	0.05	2 250.00
1 000	Feet B.M.	White oak timber and plank	40.00	40.00
4 000	"	Pine " "	25.00	100.00
246 000	"	Hemlock " "	16.00	3 936.00
755 000	"	Spruce " "	17.00	12 835.00
850	Cu. yds..	Bridge abutment culvert and receiver masonry	6.50	5 525.00
40	" ..	Coping on above	12.00	480.00
50	" ..	Vertical wall in Portland cement	5.00	250.00
1 100	" ..	" laid dry	3.00	3 300.00
16 000	" ..	Slope and pavement wall	2.25	36 000.00
2 200	" ..	Portland cement concrete	5.00	11 000.00
5	" ..	Asphaltic concrete	10.00	50.00
10	" ..	Loose stone filling	1.25	12.50
10	Sq. yds..	Portland cement pointing	0.30	3.00
44	Cu. yds..	Pavement wall	3.00	132.00
500	Lin. ft...	Cedar posts set	0.10	50.00
1 000	Lbs......	Wrought iron and steel	0.04	40.00
15 000	"	Cast-iron pipe	0.02	300.00
3 500	"	Spikes and nails	0.05	175.00
3	....	Raising bridges	40.00	120.00
3	....	Painting bridges	40.00	120.00
				$141 198.50

* These piles were at various prices.

prism of the canal, however, was not thoroughly excavated, and when that was done there was a saving of one-third in the traction necessary to draw a boat through it.

Without an engineer's knowledge of the work to be done and an engineer's plan on which to do that work, it is impossible to cope satisfactorily with difficulties which may arise. This statement is made in emphasis of, rather than in opposition to, anything Mr. Rafter has said. The relations between the specifications and the economical and possibly the successful conduct of the work is rather interesting in view of the recommendation of the Canal Committee of the State of New York, which has recently made a report on the subject. In rela-

MATE FINAL ESTIMATE ON CONTRACT NO. 4, MIDDLE DIVISION, ERIE CANAL. Mr. North.

Successful Bid.		Approximate Final Estimate.		
Prices.	Aggregate.	Quantities.	Prices.	Aggregate.
$500.00	$500.00			$1 500.00
3 000.00	3 000.00	....	....	4 895.00
0.27½	41 250.00	308 164	$0.27½	84 745.10
1.00	4 500.00	25 068	1.00	25 068.00
0.70	350.00	526	0.70	368.20
0.10	500.00	....	....	....
0.30	300.00	31 060	0.30	9 318.00
1.00	17 000.00	58 915	1.00	58 915.00
0.30	300.00	....	....	....
0.10	50.00	440 001	*	57 400.70
0.15	75.00	392 650	0.45	58 897.50
0.09	4 500.00	84 088	0.09	7 567.92
0.09	4 050.00	44 688	0.09	4 021.92
50.00	50.00	250	50.00	12.50
30.00	120.00	19 650	30.00	589.50
16.00	3 936.00	1 055 600	16.00	16 889.60
17.00	12 835.00	230 860	17.00	3 924.62
7.00	5 950.00	705.5	7.00	4 938.50
16.00	640.00	29.4	16.00	470.40
6.00	300.00	2 859.9	6.00	17 159.40
3.50	3 850.00	295	3.50	1 032.50
2.47	39 520.00	44 800	2.47	101 656.00
5.00	11 000.00	4 200	5.00	21 000.00
15.00	75.00	....	....	....
2.00	20.00	339.5	2.00	679.00
0.50	5.00	240	0.50	120.00
5.00	220.00	44	5.00	220.00
0.40	200.00	680	0.40	272.00
0.05	50.00	45 240	0.05	2 262.00
0.02	300.00	176 440	0.02	3 528.80
0.05	175.00	38 000	0.05	1 900.00
300.00	900.00	4	....	1 500.00
100.00	300.00	3	100.00	300.00
	$156 821.00			$491 076.16
		Add to this:		
		Extra work	....	53 337.17
		Force account work	....	60 982.14
		Total	....	$605 395.47
		An increase of 286% over the price bid by the contractor.		

* See p. 1084, December, 1899, *Proceedings*.

tion to the crying evil of unbalanced bids, and it has been a crying evil on the canal ever since 1836, the committee proposed to take action by making a schedule of prices, as the French do, in which the price of each item is fixed, and then allowing the contractor to bid either a discount or a premium on those figures. This would apply to all figures; thus, if the price of earth was 30 cents, and of rock 90 cents, a contractor might bid a discount of 1%, or a premium of 2%, and it would affect both the earth and the rock. The plan proposed would apparently eliminate all trouble caused by unbalanced bids, but it would be without influence on imperfect specifications, and the great expense incurred on Section No. 4 of the Middle Division was caused

Mr. North. by imperfect specifications as well as lack of engineering knowledge in handling the work. The case was atrocious. The contractors bid $3 000 to drain 6.3 miles of swamp. They were allowed to close the natural watercourses which drained into the canal and thereby turn the swamp into a pond. They knew that the banks were soft and slippery, and yet they were allowed to surcharge them with the material excavated from the canal.

The bottom of the bank on the berm side averaged 40 ft. in width, and the height was about 8 ft.; on the tow-path side the dimensions were somewhat greater. As a result of establishing and maintaining a pond of water behind this bank it was impossible to excavate the material or lay slope walls.

Instead of holding the contractors to their contract they were given an easement of more than $46 000 in the items of digging ditches, and a payment of $2 400 for pumping.

At first it may seem somewhat brutal to the contractor to say that he should have done his work at the price bid, but the case of the Chicago Main Drainage Canal might be cited, where contracts were taken with somewhat the same mental attitude as that of the contractors on Section No. 4. When the contractors said they wanted relief they were told that no relief would be granted, but that, as their sureties were abundant and satisfactory, if the work was not done by them, it would be done by the Main Drainage Commission and paid for by the sureties. After a time the contractors did the work, and not only without loss, but at a rumored profit of about 50 per cent. In relation to this work, the engineers of the Main Drainage Canal said, with pride surely, and possibly with justification, that the Chicago contractors, paying $1.50 for their poorest workmen, could have constructed the North Sea and Baltic Canal, for which the German engineers paid 75 cents for their best workmen, and have made more money thereon than the Germans.

The entire science of handling earth and rock (quicksand and hard pan being included with earth), has been advanced more materially by the attitude assumed on the Main Drainage Canal than by any other act by engineers, directors or commissioners during the preceding ten or fifteen years. The speaker thinks that if the State Engineer had said to the contractor, "You are worth more than this can possibly cost, and I will take the last cent you have and finish that work," that the work would have been done at the contract price without much loss.

The literature on quicksand is not, on the whole, voluminous, and greatly lacks defining power. Mr. Hazen's discussion probably gives the most definite information on record, and the most workable theory for quicksands which do not contain clay. But it is immediately seen that a sufficient volume and velocity of uplifting water would turn a

boulder bed into a quicksand for the time. The material, however, would become firm immediately on the withdrawal of the upward current. This cannot be asserted of a true quicksand. The relations between the included water and the earth are, in some quicksands, mysterious, and while all quicksands become stable when dry, others, particularly those containing clay, will quake after they are apparently dry. Mr. North.

The late Charles L. McAlpine, M. Am. Soc. C. E., describes a quicksand of the last-mentioned variety very fully in a paper read before the Society in 1881.* In it he says:

"Although its name conveys the idea of a mass of sand, surcharged with water until it becomes 'quick,' or susceptible of easy movement or agitation, suggesting actual life, yet engineers know only too well that this is not the most troublesome member of the family.

"The one that causes the most trouble, and is here treated of, is an argillaceous material containing no silex or grit, comminutes completely, and is usually leaden in color in its natural state, and nearly white when thoroughly deprived of water.

"So free is it from sand that it can be used with good effect in polishing or cleaning silver and the softer metals.

* * * * * * * *

"As far as possible, all traveling over the surface while being thus ditched was prevented, as it agitated the material, and caused it to retain the water more obstinately.

"After a night's quiet rest, and the great withdrawal of water through the ditches, the surface was in good condition for excavating and the material, in the words of the workmen, would then 'shovel like ashes.'

* * * * * * * *

"Care should always be had to withdraw the men and teams at once from any place which indicates that it is again becoming 'quick,' from the disturbing effect of repeated traveling over its surface.

"Nothing is gained by working longer, when this important question of rest in involved.

* * * * * * * *

"A lump of this quicksand, apparently dry, may very often be made 'quick' by a little agitation alone.

"Hard and apparently dry lumps will often become wet and pasty on their way to the dumping ground, so much so as to require additional labor to remove them from the carts."

That all of the above did not preclude, in Mr. McAlpine's mind, such quicksand as specified by Mr. Hazen, is shown by the following quotation:

"It may be assumed generally that the special mobility of such sands depends upon the presence of water filling the interstices of the mass. The mass yields to pressure in conformity to the laws of liquids or semi-fluids, varying with the degree of quickness. The degree of quickness depends upon, *first*, the gravity of the sand; *second*, upon the smoothness of the surface of the particular grains of sand; and *third*, upon the abundance of the water present with it."

* *Transactions*, Am. Soc. C. E., Vol. x, page 275.

Mr. North. Mr. McAlpine may have misnamed the material quoted, but many engineers have met with something very like it, and it is generally called quicksand.

While it cannot be doubted that quicksand may be entirely free from clay, like the samples referred to by Mr. Hazen, it seems equally certain that sand, in the usual acceptance of the term, may be wanting, or that clay which on drying becomes hard and resonant when struck can be washed from some, if not many, samples of quicksand.

Mr. Owen. JAMES OWEN, M. Am. Soc. C. E.—The speaker has held ideas on the difference between ordinary sand and quicksand, which can probably be illustrated best by a comparison between a pile of loose rock, as the ordinary sand, and a pile of cobblestones, as the quicksand. That is, quicksand is merely rounded sand, water-worn or air-worn, and, while Mr. Landreth has made a distinction by including a proportion of clay, Mr. Hazen seems to have ignored that classification.

Some years ago the speaker was asked to report on a foundation for a six-story building, of which a large proportion of the weight was to be concentrated upon one column. Sand, which was thought to be quicksand, had been encountered in the excavation, and it was a question as to whether or not it would be safe to place a large mass of concrete on it. After examining the sand the speaker reported that it was the ordinary wedge-shaped, sharp-edged sand, and that there was no fear of any flow. The building was erected, and no settlement occurred. If the speaker had found that the grains were rounded, he would not have dared to put the structure on it.

To classify quicksand in any other way than by the rounded character of its particles opens the field of speculation, and clouds somewhat the broad definition upon which engineers have depended for a number of years. For this reason the speaker believes that it would be well to determine now the difference between quicksand and ordinary sand.

The principle of the wedge-shaped sand is very forcibly illustrated by the practice of the French engineers in building one of the bridges over the Seine. The ends of the centers of the bridge were supported on large boxes of sand. The boxes had loose tops, and the longitudinal frames of the centers rested thereon. There was a faucet at the side of each box, and, when the centers were to be struck, they simply opened each faucet and the sand flowed out slowly with a certain velocity. By this means the centers were lowered very carefully and without shock to the structure.

The sand used was the ordinary cubical sand, and the vertical pressure had no effect on the flow. The sand fell on account of its own gravity alone. If the sand grains had been of rounded form, they would have flowed out with a velocity due to the pressure upon them.

GEORGE HILL, M. Am. Soc. C. E.—Some years ago, in excavating for the foundations for the *Mail and Express* Building, in New York City, the speaker encountered an extremely fine micaceous sand, designated by the contractor as quicksand. It was ascertained that the Western Union Building, adjoining, was founded on soil of the same character, and that the foundations were sustaining safely a load of about 3¾ tons per square foot. On the site of the latter building there were driven wells which had been used for water supply some years previously. After pumping for a short time the water began to appear slightly cloudy, and the building began to settle. When the pumping was stopped, the building stopped settling. Mr. Hill.

The pressure on the foundations of the *Mail and Express* Building is about 3½ tons per square foot, and the building is standing satisfactorily. There was an initial settlement, uniform throughout the entire building, of about ⅝ in., compressing the top sand, after which there was no further movement.

Within a year thereafter the speaker designed the foundation for the Pierce Building, with unit loads of 6 tons per square foot, standing on a mixed gravel and sand with rounded edges, about 18 ins. below the water-line, and with no egress for the water. That building stood satisfactorily, without any settlement.

Some years later, the Exchange Court Building on lower Broadway was erected, the excavations being carried about 5 ft. below the water-line. The character of the sand above the water-line was identical with that at the *Mail and Express* Building. As the excavation was carried below the water-line, the sand began to flow in, in spite of rather carefully driven sheet-piling for the foundation pits, the rapidity of the flow increasing with the depth. In one of the pits, about 6 ft. square, nearly a cubic yard of sand came in during the night.

It seems to the speaker that Mr. Hazen's definition of quicksand is more nearly correct than that in common use in New York City, but errs in omitting recognition of the qualification that it is not quicksand unless there is a vent for the water. That is, a material may possess all the elements of quicksand, and yet be perfectly stable and safe to use for foundations until a vent is provided for it. If such a material is called quicksand, the owner imagines that the building must sink into it, is alarmed, and harm is done; if a proper design is adopted for the foundation the material is not, and probably never will be, quicksand. Any sand which is fine is called quicksand by contractors, and they invariably claim that it contains loam. By mixing with water, shaking up, and drawing the water off, the speaker has tried to ascertain whether or not there actually was any loam or clay in much of the material called quicksand, but, so far as he has been able to see, there is no clay, but the sand is very fine. The mica-

Mr. Hill. ceous sand is excellent for foundation purposes, and, although slightly compressible, is absolutely safe if there is no vent for it. If water is present and there is an opportunity for it to flow off, even though there may not be an excess of moisture, the material will flow.

Mr. Tait. J. G. TAIT, Assoc. M. Am. Soc. C. E.—The speaker is pleased that the discussion has avoided the canal portion of Mr. Landreth's paper, and has brought out the interesting discussion on quicksand. The speaker wishes to correct any false impression which Mr. North's remarks might convey to any member not familiar with the contracts and specifications of the $9 000 000 canal improvement. Mr. North states that the contractor got $46 000 extra for bailing and draining, which he should have been made to do for the original bid of $3 000. The $46 000 was paid to the contractor at the very low excavation price of 27½ cents per cubic yard for material removed in a swamp when constructing side ditches at a loss, and these should have been estimated originally by the engineer, therefore this sum was not a present.

There was a great deal of extra work on this contract, which should, and usually does, represent some profit, but the sub-contractors, who got these supposed benefits, are bankrupt to-day. The unusually good quality of the work done, the favors demanded of the contractor, and the final non-payment for work done, through selfish, incompetent State officials, made the Erie Canal experience very costly for the majority of the contractors.

The competition in contracting to-day causes very low prices, and in a lengthy or obtuse specification, a contractor, to get the work, has to take the cheapest interpretation of what he is to be required to do, and bid accordingly. Nearly all the contractors with whom the speaker is acquainted are men who will accept the loss due to a misinterpretation, and who will do the work required by the specifications and contract, but when, in a contract calling for thirty-nine items, one alone of which, in a total of $154 000, increases from $3 000 to $49 000, and is decidedly the fault of the contract and plans, the speaker cannot understand how an honest or fair-minded man, particularly one familiar with all the conditions, can state that the contractor should have been made to do this enormous necessary amount of extra work for nothing.

Instead of taking from or injuring a contractor who has so much with which to contend, the engineer and contractor should both work together for the good of the work, the engineer seeing that he gets his money's worth, but at the same time not using his power to get something for nothing, an old-time policy not followed by all engineers.

Quicksand, with which the speaker has had considerable experience during the past twelve years, is such a bugbear that when contractors

encounter any kind of moist sand it is quite natural for them to call it quicksand, as Mr. Hill remarks. Mr. Tait.

Any sand, even the sharp angular kind, will run, even if is under a head of only one foot, but if it contains no clay, or if the grains are not rounded, it can be controlled readily by sheeting and pumping. On the other hand, if the sand is fine and rounded and contains clay, great trouble is experienced. Even in cases where the sheeting is driven far below the bottom of the excavation, this material may rise in the center and cause the sides of the excavation to cave in, thus producing excessive or unbalanced pressures on the sheeting.

Samuel Whinery, M. Am. Soc. C. E.—It would be very desirable, if it were possible, to have a correct and comprehensive definition of quicksand, but the speaker's experience has been (and he thinks it is the experience of many others), that, after settling upon what seemed at the time to be the proper definition, the very next case is likely to contradict it entirely. Mr. Whinery.

During the construction of a railroad in Western Indiana, some 30 years ago, in making a cut not far from a stream, but well above its water level, quicksand was encountered by the plows and scrapers and stopped the work at that point. In investigating in a rude way it was found that 10-ft. fence rails could be pushed down for their full length quite readily without reaching the bottom of the quicksand.

The sand was very fine, and was almost free from any admixture of clay or other foreign matter. Apparently, there was no possibility of the existence of the conditions to which Mr. Hazen refers, that is, of its being buoyed up by a rising current of water. It is true, the sand was filled with water, but this water was, apparently, in a quiescent state. There were no springs known to exist in the vicinity. The depth of the deposit was not ascertained, but it seemed to be contained in a pocket surrounded by impervious clay. The case was dealt with by beginning the excavation at the lower end of the cut, where the grade was lower, and working up the grade with deep side ditches until the deposit was reached. The water in the sand then drained out, the material became quite hard, and was then excavated with plows and scrapers. The sides of the cut were dressed to the usual slope, and they have stood quite satisfactorily ever since.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper is full of information not often found in print. Engineers are generally chary about publishing accounts of their troubles in practice, and rightly so, because the irresponsible critic is the most active of all. Mr. Le Conte.

When working in treacherous ground, such as described in the paper, the engineer is often at his wits' end to know the best way to meet existing conditions. In many instances it seems as if Nature took a malicious delight in defying all the requirements of the best

Mr. Le Conte. laid plans and specifications, based on the most trustworthy information.

The troubles depicted so graphically by the author excite the interest and sympathy of any engineer who has had the misfortune to be caught in such a trying position. The simple and effective methods adopted to overcome local obstacles are certainly highly commendable.

The laws of hydraulics and hydrostatics have been well developed by experimental investigators, but the laws governing the dynamics and statics of mud have yet to be formulated in practical shape. In June, 1826, at "Chat-Moss" on the Liverpool and Manchester Railroad, 4 miles of embankment cost nearly $150 000, and took 7 months to complete. The indomitable pluck and tireless energy of the engineer, George Stevenson, prevailed finally and left a monument to his name.

The writer was interested in a case which was particularly trying, and, at first sight, seemed to be impossible of solution.

A proprietor desired to improve a tract of marginal marshland by raising it well above the effects of tide water in a lake. The marshland was underlaid by plastic blue mud which rested upon a solid bed of yellow clay hardpan, the latter having a natural slope of about 13% toward the lake. The mud, at high-water line, was about 40 ft. deep, making the outlook rather unfavorable. The case was complicated still further by the proximity of a deep-water channel and a system of tidal sluice gates in front of, and parallel to, the shore line. This channel could not be incroached upon by pushing out the marsh mud under the pressure of the proposed new filling. The problem was, nevertheless, solved successfully as follows:

First.—A trench 4 ft. x 12 ft. wide was excavated along the irregular line of high water or outer edge of the marshland. The material taken out was deposited on the edge of the trench next to the high land, the top width of the bank being sufficient for two runways for wheelbarrows.

Second.—The work of filling in with heavy sandy material was then begun at and along the segregation line, advancing gradually toward the lake. As the filling progressed, of course, settlement commenced, and the material in the bottom of the trench began to spring up. A force of 100 laborers and excavators had all they could do to excavate and remove the material from the rising bottom as fast as it came up. This process was continued until the entire marshland was filled in solid down to hardpan, and all settlement had stopped. It is a notable fact that the total yardage thus taken out from the bottom of the trench was approximately the same as the total yardage of heavy material in the filling below the level of the marshland, namely, some 140 000 cu. yds., and yet, after the completion of operations, the dimensions of the trench were the same as in the beginning.

It would appear, therefore, that the trench simply afforded a natural vent for the escape of imprisoned mud compressed by the weight of the advancing filling deposited on the marshland. By this means these lands were filled in successfully without crowding the mud into the channel-way in front of the property to any appreciable extent. Mr. Le Conte.

Mr. Richardson.

CLIFFORD RICHARDSON, Assoc. Am. Soc. C. E. (by letter).—It may be of interest, in connection with the discussion of the subject of quicksands, to present some data in regard to the actual size of the particles in several such sands which have recently been examined in the New York Testing Laboratory by the writer.

These sands were from the following sources:

No.	Source.
	NATIONAL CONTRACTING CO., BOSTON, MASS.
11 541	Neponset Valley Sewer, Section 26, Station 3 + 50. "When wet, extremely difficult to handle."
11 542	Neponset Valley Sewer, Section 26, Station 2 + 80. "Found under peat. Very troublesome to excavate, and difficult to hold trench in line and grade."
11 544	Neponset Valley Sewer, Section 26, Station 1 + 37. "Very difficult to handle. Squeezes trench badly."
	H. P. EDDY, SUPERINTENDENT OF SEWERS, WORCESTER, MASS.
30 723	Greene Street Sewer } see *The Engineering Record*, March 24th, 1900.
30 724	" " } see *The Engineering Record*, March 24th, 1900.
30 725	" " } see *The Engineering Record*, March 24th, 1900.

For comparison with these sands, the finest ground limestone produced in a Danish tube mill, all of which passed a 200-mesh sieve, was examined.

The sands, Nos. 11 541, 11 542 and 30 725, under the microscope, were seen to be very clean, and to be made up of extremely sharp grains with no clay. Sands Nos. 11 544, 30 723 and 30 724 were equally sharp, but carried a small amount of clay, amounting to less than 1%, and not subsiding from water in a week.

The voids in the hot sand, compacted thoroughly in a 100-c. c. cylinder by shaking and tamping, were determined, where sufficient material was available, and from them the volume-weight per cubic foot. These sands were finally sifted on sieves, and elutriated by the beaker method. In this way they were divided into grades of particles of different sizes. The results are shown in Table No. 5.

Sand No. 11 541 has a wide variation in the size of its particles; consequently, it has low voids and high volume-weight. Sand No. 11 542 would probably show like characteristics, but, with a somewhat wider grading, somewhat lower voids. Each of these sands contains a very considerable amount of grains of high hydraulic value, as do the Worcester sands, Nos. 30 723 and 30 724, with voids much like those of sands used in ordinary mortar—36.7 and 34.7 per cent.

Mr. Richardson.

The most interesting sands are the two extremely fine ones, Nos. 11 544 and 30 725. They are of very uniform grading, the majority of the particles being of sizes within very narrow limits. The resulting voids are, in consequence, what are usually found under such circumstances, about 40 per cent.

TABLE No. 5.

Nos.		11 541	11 542	11 544	30 723	30 724	30 725	Finest ground limestone.
Voids in hot compacted sand		29.3%	......	40.2%	36.7%	34.7%	......	39.3%
Weight per cubic foot, in pounds		117.2	......	99.1	103.8	106.1	......	100.4
Sieve.	Diameter, in millimeters.							
....	0.035	19.2%	11.2%	65.5%	47.2%	11.6%	79.7%	63.5%
....	0.065	7.9%	14.2%	13.7%	19.6%	11.4%	9.5%	17.7%
200	0.09	18.9%	19.6%	17.8%	11.2%	9.0%	9.8%	18.8%
100	0.17	34.0%	22.0%	3.0%	13.0%	39.0%	1.0%	....
80	0.23	11.0%	8.0%	1.0%	4.0%	12.0%	......	....
50	0.31	7.0%	16.0%	......	3.0%	10.0%	......	....
40	0.50	1.0%	4.0%	......	1.0%	3.0%	......	....
30	0.67	1.0%	2.0%	......	1.0%	3.0%	......	....
20	1.00	....	2.0%	......	....	1.0%	......	....
10	2.00	....	1.0%	......	....	....	......	....
Greater than	2.00	....	2.0%	......	....	....	......	....

The sizes of the grains of these sands are smaller than those in the ground limestone dust, that is to say, than the finest Portland cement.

Mr. Hazen's claim that quicksand depends more for its peculiarities upon the size of the particles and upon their small hydraulic value than upon any other characteristics, round shape of grain, presence of clay, etc., seems to be confirmed.

It would be of interest to examine the quicksands encountered in the Erie Canal in the same way, and the writer would be glad to do so if samples from that locality, and from any others where such sands are met, can be furnished.

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PAPERS AND DISCUSSIONS.

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THE EXACT DESIGN OF STATICALLY INDETERMINATE FRAMEWORKS. AN EXPOSITION OF ITS POSSIBILITY, BUT FUTILITY.

Discussion.*

By Messrs. H. GOLDMARK, GUSTAV LINDENTHAL, C. W. RITTER, W. DIETZ, JOSEPH SOHN, G. JUNG and F. H. CILLEY.

HENRY GOLDMARK, M. Am. Soc. C. E.—In this interesting paper, the author applies the well-known principle of Virtual Velocities to a study of the stresses and deflections in trusses with redundant members and other structures not statically determinate. Mr. Goldmark.

As the result of his investigation, he arrives at the conclusion that such constructions show no advantage, either in economy of material or in rigidity, over the simpler determinate forms, while even the best methods for computing their stresses are extremely laborious and of doubtful accuracy.

He therefore recommends that all indeterminate forms, such as trusses with a multiple web system, continuous girders, arches with less than three hinges, suspension bridges with continuous stiffening girders, etc., should be abandoned entirely in practice.

As to the method of computation which he has used (that of Vir-

* This discussion (of the paper by Frank H. Cilley, S. B., printed in the *Proceedings* for October, 1899), is printed in *Proceedings* in order that the views expressed may be brought before all Members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to May 25th, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Goldmark. tual Velocities or Least Work), he considers the amount of numerical labor involved so great as to make the method practically valueless.

While the speaker is by no means an advocate of complex truss forms where there is no good reason for using them, he is fully satisfied that there are many cases in practice where these tabooed forms give by far the best results.

As a matter of fact, the simplicity of the strain sheet is, in many cases, by no means a proper criterion as to the excellence of the design. Thus, in railroad bridges of moderate length, the tendency of late years, as the result of actual experience, is away from the "ideal" single-intersection bridge with alleged frictionless pins. Such spans are now almost universally built with riveted connections, and, in many cases, with multiple systems of webbing. Even in larger bridges, the floor system is almost invariably riveted between the posts, though the transference of loads to the main trusses is thus made less direct. In all these cases intricate secondary stresses are introduced, but the advantages obtained are believed to counterbalance this theoretical drawback.

In the case of the larger arches and suspension bridges the introduction of hinges at the center is properly considered to break the continuity of the floor system and wind bracing, to add to the complication of the details, and in all probability to decrease the rigidity of the whole bridge under moving loads.

It is true that the absence of hinges makes the computation of stresses more difficult and somewhat less certain, but it is believed that with the homogeneous material now used, and the excellent workmanship of our shops, perfectly safe structures can be built without any excessive cost. Besides this, there is, in the speaker's mind, little doubt that even the best designed hinges or pins can hardly be counted upon to act as theory requires.

But besides such bridge forms as those above mentioned, in which the designer is at liberty to choose between so-called determinate and indeterminate forms, there are many engineering structures which are, of necessity, of a complex nature. Such are the frame works for the enclosing of large shops and auditoriums, in which the roof trusses and the steel posts which support them are connected by knee-braces or other connections, and many forms of cranes and derricks; furthermore, metallic gates for canal locks, as well as ship caissons for dry docks and harbor works. In all these constructions some method of computing indeterminate stresses is desirable.

In such cases, as well as for computing bridge stresses, the speaker has found the Method of Virtual Displacements (Method of Least Work) of great value, while he has not found it excessively laborious, particularly if it is used as a means of devising approximate methods

of sufficient exactness in each case rather than for computing individual designs. Mr. Goldmark.

A brief reference to the bibliography of this method of computation may be of some interest.

The first application of the principle of Virtual Velocities to the determination of deflections and stresses in frameworks is commonly ascribed to Clerk Maxwell, whose paper embodying this method appeared in the *Philosophical Magazine* for 1864. The same theorem, in a slightly different form, was, however, used independently, to solve problems of this class, by M. Menéhea,[1] in 1858, and Professor Lamé,[2] in 1866.

The further systematic development of the method is, however, due mainly to German writers. The earlier articles of Professors Mohr[3] and Winkler[4] gave an application of the method to a variety of structures, such as continuous girders, trusses with multiple web systems, arches with less than three hinges, etc. The most complete exposition of the more recent German methods, however, is given in the works of Professor H. Müller-Breslau, of Berlin. His special treatise[5] on the subject the speaker has found of great assistance in practice, and he would consider an English translation of much value for American engineers.

Reference should also be made to a monograph[6] in French, by the late M. Castigliano, a young Italian engineer of great promise, who gives a very elegant analysis of the "Method of Least Work," with many applications.

In the English language, the valuable paper by George F. Swain, M. Am. Soc. C. E., published in 1883,[7] is the only extended article on this subject known to the speaker, though brief expositions are given in some textbooks on applied mechanics.[8] Mr. Cilley has therefore conferred an obligation, on English-speaking engineers, by his interesting study on the subject.

Gustav Lindenthal, M. Am. Soc. C. E. (by letter).—This paper is a contribution to the old controversy as to whether or not statically determinate structures are superior to statically indeterminate ones, and is a scholarly attempt on the affirmative side of the question. Mr. Lindenthal.

Mr. Cilley, like others before him, advances as principal reasons in favor of determinate designs, their alleged economy, the greater certainty of their calculation, and the greater accuracy, therefore, in the dimensioning of the cross-sections.

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[1] *Comptes Rendus*, 1858, Vol. xlvi, p. 1056.
[2] "Leçons sur la Théorie Mathematique d'Elasticité des Corps Solides."
[3] *Zeitschrift des Architekten und Ingenieur Vereins zu Hannover*, 1874 and 1875.
[4] *Zeitschrift des Architekten und Ingenieur Vereins zu Hannover*, 1879.
[5] "Die neueren Methoden der Festigkeitslehre." Berlin.
[6] "Théorie de l'Equilibre des Corps Solides." Turin, 1879.
[7] *Journal*, Franklin Institute, February, March and April, 1883.
[8] Cotterill, "Applied Mechanics," p. 551.

Mr. Lindenthal. Granting, that the last two arguments have much to commend them, it is nevertheless true, as regards economy, that diminutive differences in weight of metal, as shown by strain sheets of one and another type, will rarely be the sole deciding element in the choice of designs. Other considerations than mere economy of metal influence greatly the cost and merit of a structure. But in any case, strain sheets to be comparable should be complete, and include all strains which affect the sections. This does not seem to the writer to be the case with the strain diagrams in the paper. Before indicating in what the incompleteness is believed to consist, the preliminary question is justified:

"On what theoretically equivalent basis (aside from same length of span and moving load), should a comparison of the economy of determinate with indeterminate arch types be made? Should it be on the basis of the same figure of inclusion, as the author has done, or on the basis of congruous equilibrium polygons under some assumed load, as would undoubtedly be the case for ribbed arches, or for suspension bridges, which are nothing but inverted arches?"

The short span of primitive figure, chosen by the author with only 10, respectively 11, members in the frame, serves to illustrate the great labor of a detailed analysis for each member of the indeterminate arch; but in all other respects it is misleading. If the diagrams of the two types were each enlarged from 4 to, say, 40 panels, then the determinate figure of the author would become the form known as the Eads arch, for which a certain depth at the quarter is economic, and the indeterminate figure would become a sickle arch, for which another depth at the center is economic. The enlarged diagrams, however, would no longer have the same figure of inclusion. Only one chord, namely the upper one, would coincide in both, but that is not enough to furnish equivalent conditions of economy.

Suppose a comparison were made between single (determinate) and continuous (indeterminate) trusses. Obviously, we would start out with the same ratio of height to span (at center of span) for each kind, and compare the strain sheets and material required for them on that basis, to come to a correct conclusion of their economy. Similarly, a comparison of arches will have to proceed. We cannot compare fairly the economic merits of arches of different rise, or of an arch having a ratio of rise to span of three-tenths with another arch having the same ratio as to one chord only and another much smaller ratio as to the lower chord, as Mr. Cilley does. He compares two primitive figures, one statically determinate, and the other made indeterminate by inserting a redundant member. Considered as generic forms, the figures are dissimilar and not comparable. His conclusions as to the economy, or the want of it, in indeterminate forms, arrived at in this manner, are wholly irrelevant and inapplicable to long spans, the only ones for which arch types are economically used.

This will appear more clearly when we consider that the closed frame of the author, with a depth of one-eighth of the span at the center would in practice occur as a sickle truss which is determinate, but hardly as a sickle arch which is indeterminate, and with that depth of rib, would be wasteful. No arch bridge has been built yet with a depth of rib of more than one-sixteenth. More frequently it is less than one-thirty-second of the span. Mr. Lindenthal.

As the temperature strains for the same arch form diminish directly with the ratio of rib to span, they are, in existing bridges, very much smaller than those given by Mr. Cilley on page 616.* His maximum is $\pm$ 45% of the full load for $\pm$ 100° Fahr. In the largest sickle arches thus far built, namely, that of the Garabit Viaduct in France and of the Gruenenthal Bridge over the Baltic Canal in Germany, the maxima are under $\pm$ 18%, when corrected to the same extremes of temperature.

But, as stated before, a comparison between forms of the general characteristics of sickle arches and Eads arches, simply because they happen to coincide as to one chord, can lead to no rational conclusions.

Comparisons of carefully worked out designs of different arch types, made by him and others, have convinced the writer that, other things being equal, the indeterminate types are invariably more economical. This is true even if the assumption that there are no temperature strains in three-hinged types were correct. But this assumption is not correct as shown by the writer long ago.† That time-honored theory, found in all text books, is based on error. Three-hinged arches are not only not free from temperature strains, but for certain conditions such strains are just as large as in arches of the two-hinged type. The difference is merely in their distribution. The supposed advantage in that respect of the three-hinged over the two-hinged arch is illusory.

Professors Merriman and Jacobi's "Higher Structures" is the only work, so far, referring to the writer's theory on this subject. But although mentioned there merely in connection with three-hinged stiffening trusses in suspension bridges, it applies, nevertheless, to all forms of arches with three hinges, whether of the ribbed, spandrel-braced, or of the erect or suspended form. Each is, necessarily, differently affected; but in all cases the strains are due to the changes in the curvature of the equilibrium curve from dead load for different temperatures.

If the author will trace the strains, due to the falling and rising of the center hinge from changes of temperature, he will find them by no means of negligible smallness, even for the large ratio of rise to span in his inconclusive figure.

* *Proceedings*, Vol. xxv.

† *Engineering News*, 1888, p. 174; and "Appendix to Report of Board of Engineer Officers as to Maximum Span Practicable for Suspension Bridges," 1894, p. 68, and elsewhere.

Mr. Lindenthal. The temperature strains are particularly large in flat arches, and, to repeat it, the insertion of a center hinge does not eliminate them. It only serves to remove the maximum bending moment from the center to each quarter. So that where the two-hinged arch has one maximum bending moment, the three-hinged has two of the same intensity in most cases.

The new Alexander III Bridge in Paris, for instance, having a rise of only one-seventeenth of the span, is thus far the flattest metal arch attempted. It is of the three-hinged type, here properly chosen because a small depth of rib was necessitated at the center by the prescribed clearance above the river. The writer has not seen a strain sheet of that bridge, but believes that in accordance with the common theory, the arches have been assumed to be free from temperature strains. As a matter of fact, the bending moments at the quarters, from temperature changes, are fully as great as at the center of a two-hinged arch, having the same depth throughout as at the quarters (where the rib is one seventy-second of the span).

If the Washington Bridge in New York (510-ft. span), or the Niagara Falls road bridge (840-ft. span), which are both of the ribbed-arch type, had been built with three hinges instead of only two, the extra metal required to meet the temperature strains would have been the same as for two-hinged arches, with this difference, that the additions to the chord sections would have been largest at the quarters instead of at the center, and the additions to the web members largest at the ends of each half arch, instead of at the ends of the whole arch. In spandrel-braced arches, the center hinge may be of value, as here it really reduces the intensity of temperature strains, but does not eliminate them, by any means.

The law applies also to stiffened suspension bridges. The well-known Point Bridge at Pittsburg (800-ft. span) has three hinges; the bracing is above the chains. The bridge has been believed to be free from temperature strains, which is not the case. The maxima from temperature are those corresponding to the maxima at the center of a suspended two-hinged arch, having a rise of one-eighth of the span, and a depth of rib of one thirty-second of the span. But in this case the maxima in the chords, which in the two-hinged arch occur only at the middle of the span, reach, with little variation, from hinge to hinge, while the web is only slightly affected. On the whole, this type, and the so-called Fidler type, are the worst forms for temperature strains, and yet both are determinate forms. The suspended end spans of the Tower Bridge in London, which are of the Fidler type, are certainly exposed to bending strains from temperature changes, although the designers believed them to be eliminated by the insertion of middle hinges.

That the neglect to provide for temperature strains in three-hinged

arches has not yet endangered such bridges proves anew how useful the so-called factor of safety is, and how true its designation as a factor of ignorance. Mr. Lindenthal.

A second correction in the strain sheets, for a proper comparison of the two types treated by Mr. Cilley, is required for the large bending moments resulting from the immobility of the hinges, incorrectly ignored by him. In none of the existing arch bridges has a turning motion at the hinge ever been observed. The friction is too great. But the motion is not even desirable. There could be no motion without wear, which after a while might affect the strains more than the immobility of the hinges.

The bending moments from that cause cannot be regarded as mere secondary strains of local effect, because they make themselves felt throughout the entire arch frame, and in flat arches or shallow ribs require considerable additions to the sections. It is true that the moments can be much reduced by using for the hinges small and long pins rather than short and large ones, for affording the necessary bearing area. But in any event there are, under equivalent conditions, twice as many bending moments from that cause in three-hinged as in two-hinged arches, while they are absent entirely in hingeless arches, in which the temperature strains are largest.

The present theory of metal arches requires, therefore, the following important corrections:

Three-hinged arches: Present theory, statically determinate system, no temperature strains.—Corrected theory, statically determinate system, temperature strains plus bending strains from three hinges.

Two-hinged arches: Present theory, singly indeterminate system, plus temperature strains.—Corrected theory, singly indeterminate system plus temperature strains, plus bending strains from two hinges.

Hingeless arches: Present theory, two-fold indeterminate system, plus temperature strains; requires, of course, no correction, inasmuch as there are no hinges.

Strain sheets prepared on the corrected theory, for the same rise of equilibrium curve for similar loads in all three types, are alone fairly comparable as to economy of metal. The three-hinged and two-hinged types require additions to the cross-sections of the members over those obtained on present theory. These additions cannot be ignored. When made, the two-hinged arch will in every case show great economy over the three-hinged arch.

The secondary stresses, which occur at the panel points from the elastic deformation of the frame, affect in practice only the details and strength of connections. They are not different from those in trusses, cantilevers, and other frames. They may be ignored in a comparison of strain sheets.

The objection to indeterminate structures cannot very well be

Mr. Lindenthal. based on the ground that the strains cannot be determined with the same exactness as in statically determinate structures, because such is not necessary, either for safety or economy.

The dangerous strains, their limits, and the conditions under which they occur, can always be ascertained with sufficient accuracy without the endless drudgery which, as Mr. Cilley shows, a detailed analysis for each member of a large structure would involve. There is a limit to the refinement of the engineer's calculations, beyond which nothing of practical value can be gained.

With the utmost accuracy of computation, in statically determined or undetermined structures, we will never be able to dispense with a three to ten-fold safety, as the case may be, to cover defects of manufacture, want of uniformity in the material, and the many petty indeterminate, as well as undetermined, stresses which are always present. The minimum and maximum limits of stresses ascertained, their frequency properly judged and provided for, we need have no anxiety for the safety of statically indeterminate structures.

No failures of bridges, caused by their indeterminateness, have yet occurred. Some, over 40 years old, are still carrying safely railroad loads, which, in the meantime, have largely increased. The greater diffusion of the shearing strains, particularly characteristic of indeterminate structures with multiple systems of web members, has prolonged their life. Repairs in a few cases are known to have been necessitated by poor details. Many statically determinate structures, built at the same time, and proportioned for the same unit stresses, have shown less resistance to wear. Some American railroads have already the third generation of metal bridges.

It is not irrelevant to the subject under discussion to refer here to the wide-spread bias against the very useful continuous girder, which is alleged not to be economical. This is true only when the chord members, subject to alternate stresses far within the elastic limit, are required to be largely increased in accordance with the discredited Launhardt formulas, or with other similar rules, unjustified by sound reasoning. We have no proof whatever of the strength of iron or steel being affected by alternating stresses within half the elastic limit. The increase of section, as required by bridge specifications in vogue, for members, subject to such strains, is a waste of money and material. The good results with the very economical, durable and rigid, continuous girders, proportioned and built before the present rules of dimensioning were known, ought to have weight with thinking engineers.

That the writer is not alone in his condemnation of the modern rules for alternate stresses in bridge construction, is gratifyingly evident from the report of the discussion at the International Railway Congress held in London in 1895.*

* Congress Bulletin, page 79.

"But," quoteth the theoretician, "consider the dangerous strains in continuous girders, if a pier should settle, or when iron towers expand in height!" The answer is, that piers should be built so that they will not settle more than a certain allowable amount, and that the variations in the levels on iron towers can be allowed for readily in the dimensioning of the girders. This is a part of engineering science, and surely what the Chaldean and Roman bridge builders accomplished in the way of good foundations, the modern engineer should at least be able to equal. And when secure foundations cannot be had, neither continuous girders nor other forms of indeterminate structures should be chosen. It is the engineer's business to investigate and to discriminate. Mr. Lindenthal.

The shaky and vibrating cantilever structure, so much affected as a great improvement in that respect, is by no means an adequate substitute for the rigid, compact and economical continuous girder, properly designed.

Another point deserves attention. The author seems to ascribe much importance to deflections as a means of judging the rigidity of a bridge. What is a rigid bridge? If we take rigidity in metallic bridges to mean absence of vibration, then deflections are a deceptive criterion.

The public considers a bridge which vibrates, as an inferior structure; and one which does not, as a superior one. This ought to be the guide, also, to the engineer. Sometimes, structures regarded as rigid have great deflections and little vibration, and *vice versa*, of which the following are a few instances:

The old Niagara single-track railroad suspension bridge (indeterminate) deflected ordinarily 10 ins., under a train, but a pedestrian on the roadway below would hardly notice it and would feel very little vibration. It was a more rigid structure than the double-track cantilever railroad bridge (determinate) located near by, which deflects ordinarily only 3 ins., but a pedestrian on it feels disagreeable sensations, and can hardly keep his feet when a train passes over the bridge.

The St. Louis arch bridge (indeterminate) vibrates very noticeably under every train and team, while the Merchant's Bridge (determinate) located near by is a fairly rigid truss structure.

The unsightly Market Street cantilever bridge (determinate) in Philadelphia is unpleasantly known for its spring-board motion, while the old cast-iron arc bridge (indeterminate), located below, at Chestnut Street, is rigid. One of the abutments of the latter bridge has yielded about 3 ins., but none of the calamities predicted in such cases for indeterminate arches have occurred. The behavior of three famous steel-arch bridges is also instructive, viz.: The Washington arch-bridge (indeterminate), in New York, has ribs with solid webs; its deflections from ordinary loads are nil, but its vibrations, even

Mr. Lindenthal. under a single-horse truck, are so noticeable that they have been the subject of (of course unfounded) anxious communications to newspapers. If it had three hinges instead of two, the vibrations would be worse. The Niagara Falls ribbed arch bridge (indeterminate), completed two years ago, is similarly complained of, while the spandrel-braced railroad-arch bridge (indeterminate) below is one of the most rigid metal bridges in existence.

Specially interesting, in that respect, are also four different types of large bridges at Buda Pest.

The railroad bridge with continuous lattice girders (indeterminate) is rigid under fast trains; the famous Buda Pest suspension bridge, with no stiffening to speak of (and therefore determinate), shows only little less vibration than the heavy Margarethen spandrel-braced, arch bridge (indeterminate), or the new cantilever bridge (determinate) near by.

Owing to the prejudice against indeterminate structures, most of the very high, iron-trestle viaducts in this country are subject to so much vibration, that trains must greatly reduce speed over them. With girders cut over every post, and provided with slide bearings, as demanded by the usual bridge specifications, sufficient rigidity cannot be obtained. The high viaducts, with continuous girders (indeterminate), in Europe, some of them over 40 years old, are, on the other hand, fairly rigid under fast trains.

Compare the Forth Bridge (indeterminate), proportioned for a live load of 4 000 lbs. per lineal foot, a riveted cantilever structure, in which 15% of the metal is in the strong lateral bracing, with the Brooklyn Suspension Bridge (indeterminate), proportioned for only 2 000 lbs., having no lateral bracing to speak of, and a very defective stiffening system. The latter bridge is subject to much greater deflection than the Forth Bridge, but both bridges are equally free from noticeable vibration under the loads and at the speeds for which they are designed.

The remarkable rigidity of even scantily stiffened suspension bridges and the reasons therefor have been discussed by the writer on former occasions. It is his opinion, that for long spans properly stiffened suspension bridges are the most rigid of any metal type.

Thus the study of existing bridges, of both the determinate and indeterminate kind, affords better instruction in rigidity than the mere comparison of deflections. Naturally, these ought to be always as small as possible.

The writer does not advocate indeterminate structures, but neither is he prejudiced against them. A decision as to their true economy and merits can be reached only from case to case. It seems to him that the sweeping conclusions of Mr. Cilley and of others against them are founded on incomplete investigations and are at variance with known facts.

Prof. C. W. RITTER* (by letter).—In his detailed comparison of statically determinate and indeterminate frameworks, the author endeavors to demonstrate the decided superiority of the determinate forms. He claims that the determinate structures are, theoretically, more economical than the indeterminate ones. He shows that in indeterminate structures, a slight inaccuracy in the length of a member or a slight change of temperature, can effect a considerable and even serious change in the stresses. He emphasizes the fact that discrepancies in the levels of the supports of continuous girders, or a slight yield of piers or abutments of two-hinged and hingeless arches, have considerable effect on the interior stresses. He also points out the laboriousness of the exact design of indeterminate structures. Mr. Ritter.

It will scarcely be necessary to concede that all these arguments are doubtless very reasonable, and the writer thinks it wise to emphasize now and then the advantages of determinate structures and to expound the uncertainties of the basis on which the design of indeterminate structures is founded.

In German literature, these questions have often been discussed and analyzed. A thorough investigation of the matter has not been without conclusive results. The use of the multiple intersection system has commonly been dropped; arches which, formerly, were all designed without hinges are now very often provided with two or three of them. Within the last ten years, even several stone arches have been built with three hinges. The cantilever system has been frequently substituted for the system of continuous girders.

Although a noticeable change has taken place in the manner of constructing frameworks, European engineers have not, as yet, abandoned indeterminate forms entirely, and the writer dares to say they never will. Without disputing the advantages of the determinate forms, a thorough and careful examination of the question leads to the result that, considered from the practical standpoint, the indeterminate frameworks must frequently be declared preferable, at least for European practice.

First, it may easily be perceived that some of the above-mentioned disadvantages of the indeterminate structures are, in many cases, of little weight, and that some others adhere also to the determinate forms. If the foundations are absolutely solid, why should we not build a continuous girder, as well as a cantilever. The influence of varying temperature on two-hinged and hingeless arches decreases with increasing height for the same span, and, in case such arches have other advantages besides, we should not hesitate to prefer them to the three-hinged arch. Unequal warming of the members of an indeterminate framework exerts indeed a disadvantageous influence on the stresses, but the same may be said of the one-sided warming of the

* Professor of Civil Engineering, Federal Swiss Polytechnic, Zurich.

Mr. Ritter. members of a determinate structure. In fact, we will never succeed in avoiding entirely the influence of varying temperature.

In regard to economy, the author asserts that the determinate structures need less material than the indeterminate ones. For this assertion he gives a plain and convincing mathematical proof. But let us be careful in applying this result to our practical constructions.

For instance, it can be shown that the framework designated *a*, Fig. 18, which is determinate, requires more material than the framework designated *b*, which is indeterminate. Assuming each panel load to be equal to 4, and that, by introducing initial stresses, the diagonals in each panel in form *b* are equally strained, the stresses indicated by the figures will result. Multiplying these stresses by the lengths of the respective members, we have:

Form *a*—	Upper chord	$= 2\,(10 + 16 + 18)$	$= 88$	248
	Lower chord	$= 2\,(10 + 16)$	$= 52$	
	Posts	$= 10 + 6 + 4 + 6 + 10$	$= 36$	
	Diagonals	$= 2\,(14.14 + 8.49 + 2.83)\,\sqrt{2}$	$= 72$	
Form *b*—	Chords	$= 2\,(10 + 13 + 17 + 13 + 17)$	$=140$	232
	Posts	$= 7 + 2 + 2 + 2 + 7$	$= 20$	
	Diagonals	$= 2\,[14.14 + (2 \times 4.24) + (2 \times 1.41)]\,\sqrt{2}$	$= 72$	

Nevertheless, the assertion of the author is correct, inasmuch as there exists a third form *c*, included in *b*, which requires still less material, for we have:

Form *c*—	Upper chord	$= 2\,(10 + 12 + 16)$	$= 76$	224
	Lower chord	$= 2\,(14 + 18)$	$= 64$	
	Posts	$= 6 + 6$	$= 12$	
	Diagonals	$= 2\,(14.14 + 5.66 + 2.83 + 2.83)\,\sqrt{2}$	$= 72$	

At the same time, we recognize that the inner posts have disappeared. As these posts, as a rule, are necessary for attaching floor-beams, the form *c* is in most cases practically useless, which proves that the advantage of the determinate frameworks, as far as the theoretically required amount of material is concerned, may, by accessory circumstances, easily be reversed into a disadvantage.

The author himself shows that for varying loads the above-mentioned rule loses its validity. There are still other facts which often prove the indeterminate structures to be more economical.

First, in using intersecting diagonals, we obtain the advantage that in these members the stresses are halved, and, in consequence, the struts can be attached directly to the chords; while, in the contrary case, we are obliged to use costly connection plates. Again, by intersecting the ties with the struts, we shorten the distance of supports

of the latter. Further, it is a common experience, that the stresses in the various members of an indeterminate framework are of greater uniformity than those of a determinate one, a fact which naturally leads to some economy. Finally, for cantilevers and three-hinged arches, the cost of the structure is increased by that for the hinges, and it is not unusual that this additional cost exceeds all economy secured otherwise through statical determination. It may be remarked also that the connections of the lateral system, on account of its continuity past the hinges, very often lead to clumsy details and difficulty in construction. Mr. Ritter.

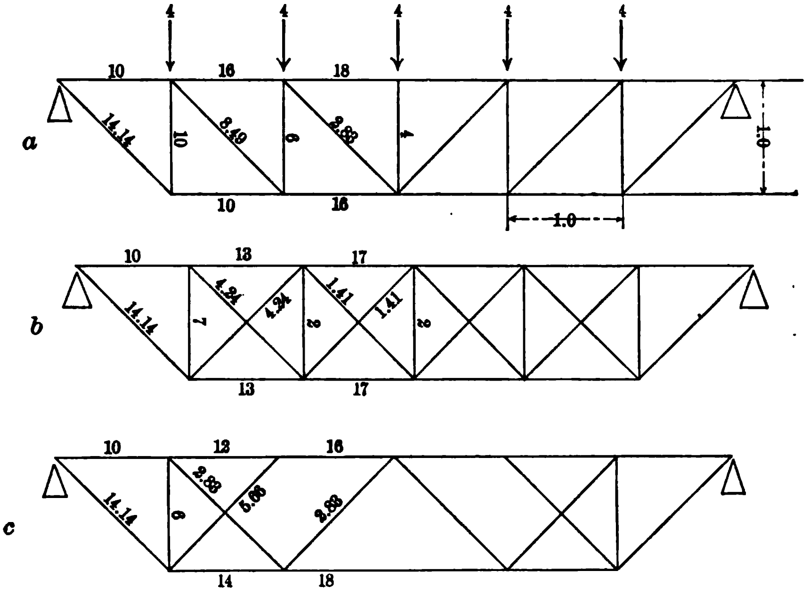

FIG. 16.

After all, it will be difficult or even impossible to predict which of the two systems will prove cheaper in the end, without computing and designing, in each special case, both of them. The greater labor involved in computing statically indeterminate structures can rarely be a reason for abandoning them. The methods for their computation have been perfected to such a degree that the time required for it alone is hardly of any moment in comparison with that required for the entire design.

One of the chief objections, emphasized by the author, to statically indeterminate structures is the possible discrepancies in the length of members. This objection is an important one for the United States of North America, where pin-connected structures are generally built.

Mr. Ritter. It is of less importance in Europe, where riveted connections are used more generally. If the length of any member of an indeterminate, pin-connected framework is inaccurate, the defect, as a rule, cannot be corrected easily, while in riveted connections such deficiencies are compensated by the red-hot rivets which will fill out the holes even if these, within certain limits, do not match. European specifications, not unlike American ones, strictly provide that if, in connecting parts or members, the rivet holes fit badly, the holes have to be enlarged and larger rivets have to be used. Stretching the bars, in order to match the rivet holes, is strictly forbidden. If in spite of these precautions a slight discrepancy remains, it is, as mentioned above, compensated by the rivets themselves, which, if properly driven, will fill out the holes perfectly. It seems to the writer that for this reason the inducement for avoiding statically indeterminate structures exists far less for rigidly rivet-connected than for pin-connected structures.

Regarding stiffness, the author gives us an interesting comparison between the deflection of a two-hinged and a three-hinged arch. He finds that the difference is very insignificant. It seems to the writer that this result, for the example in question, has to be attributed to the fact that the theoretical height of the two-hinged arch is actually less than that of the three-hinged one. In general, structures with hinges (arches or cantilevers) are less stiff than those without hinges. For instance, the fixed span of a cantilever deflects exactly as much as a single-span girder, while the deflection of a continuous girder of the same span is from 30 to 50% less. Let it be remembered that in railway bridges such hinges often produce shocks, which benefit neither the bridge nor the rolling stock.

In reference to the stiffness of bridges, we should not only consider the elastic deflections under varying loads, but also the vibrations of the structure in a vertical and lateral sense. It can scarcely be contended that, in this respect, statically indeterminate structures are safer, although, as the author says, a strict proof for this assertion can scarcely be given. Now, is not the extensive adherence of American engineers to stiff joints in upper chords, as well as the rigid attachment of floor-beams, stringers, and lateral, portal and sway-bracing, a proof of their sound constructing sense, although all these construction details clearly involve statical indetermination? The author, it is true, refers principally to vertically loaded frameworks, but what is sound for floor beams, stringers and bracing, is not likely to be wrong for frameworks throughout.

The author should be thanked for his intelligent investigations and his manifold suggestions. In any case, it will be unreasonable to build an indeterminate framework where a determinate one is just as advantageous, and the author's endeavor to substitute determinate for indeterminate forms is praiseworthy. But on the other hand, it is the

writer's opinion that indeterminate structures are in many cases preferable, and that we will never be able to do without them entirely. Their frequently greater economy and their greater stiffness, as well as their adaptability to various circumstances, are such important advantages, that it would not be wise to throw them overboard on mere principle. On the contrary, it should be regarded as the task of engineering science to study the qualities, the peculiarities and especially the deficiencies of indeterminate structures, to search for necessary remedies, and to simplify further methods of computation. Mr. Ritter.

Prof. W. DIETZ* (by letter†).—The writer was a pupil of Head-Master H. Gerber, who, by the introduction of continuous girders, such as cantilevers and those simply supported at the piers, has gained great distinction in the art of bridge building, and who, as is perhaps not generally known, by the construction of the street bridges over the Main, near Hassfurt,‡ furnished the prototype for the truss system of the Firth of Forth Bridge. The writer, therefore, has had the opportunity of becoming intimately acquainted, both theoretically and constructively, with the greatest variety of statically determined truss systems. It follows, therefore, that he has retained a certain preference for such systems, with their strains so clearly and quickly obtained. Yet his vocation makes it necessary for him also to investigate, thoroughly, both theoretically and practically, statically indeterminate systems of the most different kinds, and his experience gained thereby, together with the indisputable advantages of undetermined systems under certain conditions, compel him not to withhold due acknowledgment thereof, and to protest against unfounded attacks upon the existence of these trusses. Mr. Dietz.

The writer, with a large amount of material at his disposal, must characterize the author's closing sentence "that the use of indeterminate framework is certainly not advantageous from an economic standpoint, nor apparently from any other standpoint," as not in accordance with facts, and hardly capable of demonstration.

The matter is not as simple as the author represents it to be. As is well known, it is not only the length of span between supports that influences the volume of iron in bridge trusses, but also the greater or smaller number of panels into which the span is divided, the length and form of cross-section of the compression members; the kind of joints, and the manner in which they are made, as also the riveting of adjoining parts; and, finally, to a very great extent, the method used in dimensioning. As regards the latter, the method by Gerber, as specified in Bavaria, which takes into account the impact of variable loading, gives considerably different cross-sections in individual

* Technical Highschool, Munich.

† Translated.

‡ *Zeitschrift für Architektur und Ingenieurwesen*, 1898, p. 563, Fig. 1.

Mr. Dietz. members from those determined by the widely used Launhardt-Weyrauch formula, which does not consider the effect of impact.

In order, therefore, to make a professional comparison, in regard to the amount of iron needed, between several different truss systems, it is necessary to take account of all the foregoing factors, and to observe the condition that all the trusses are dimensioned by the same method.

Most of these factors are taken account of by introducing the so-called construction coefficient with which the theoretical volume of the different members must be multiplied in order to obtain the practical volume. The determination of these coefficients is, however, a work of considerable scope, and leads to results of practical value only when based upon a large number of data, which the bridge companies, for reasons which are obvious, do not give out generally.

Further, it should not be overlooked that the superstructure of a bridge forms only a part thereof, and that not only the metal construction, but also the abutments and piers, the method of erection and the maintenance of the structure, should be taken into consideration, so that in special cases it is quite possible that a considerable increase in the cost of the iron structure may result in a saving on the final cost of the whole bridge. The ultimate object of all our technical information and knowledge is to arrive at the lowest possible total cost, providing at the same time for an even strength of all the parts.

There is ample proof to show how often the effort is made in Germany, even in extraordinary cases, to use statically determined truss systems, as, for instance, in the Neckar Bridge, in Mannheim* and the Donau Bridge in Budapest.† If, however, in a large number of structures‡, built of late, indeterminate frameworks have been used in the main truss systems, the reason for this should not be sought in a one-sided preference for difficult theoretical problems, but in the fact that after a careful consideration of all circumstances in each case, these systems were found to be the most advantageous.

As a particularly instructive example of this class, the Müngstener Viaduct§ can be mentioned. In this case the most thorough comparative studies regarding the effects of brakes, and the matter of erection, led finally to the selection of a combination of trestle bridge with a statically indeterminate arch with fixed ends.

It should be noted, in this connection, that very interesting experiments were made upon this bridge, and that they showed a very satisfactory agreement between the theoretical and the actual strains in the truss members.

* *Zeitschrift des Vereins Deutscher Ingenieure*, 1891, p. 89, Fig. 12, and *Zeitschrift für Architektur und Ingenieurwesen*, 1898, Plate xiv, Fig. 11.

† *Zeitschrift des Vereins Deutscher Ingenieure*, 1894. International Competition, p. 1238, Fig. 72

‡ *Zeitschrift für Architekture und Ingenieurwesen*, 1898, p. 561.

§ *Zeitschrift des Vereins Deutscher Ingenieure.* "The Müngsten Viaduct." p. 1321.

Mr. Dietz

As is well known, the Germans are much interested in the thorough investigation of the so-called secondary strains* which are found in every truss of whatever system, arising either from the inflexible, riveted joints, or from the frictional resistance of the pins, which, even in the best constructed, pin-connected bridges must be overcome before an actual turning of the individual members about the pin can take place. From this point of view the two-hinged arch, for instance, is, when properly designed, considerably superior to the ordinary truss bridge resting on two supports; and this is an additional factor which in connection with a pleasing outer appearance, which the Germans happily are appreciating more and more, has influence in determining the choice of a truss system.

In the proceeding, the writer has outlined the different elements which should be considered in judging the merits of different truss systems; he would now, in conclusion, prove by figures how little permissible it is to draw general conclusions as to the economical

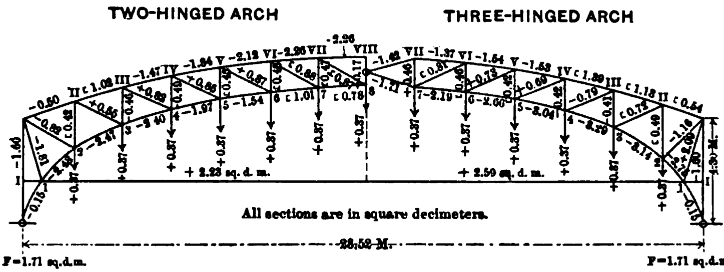

FIG. 19.

superiority of the three-hinged over the two-hinged arch, even in purely theoretical respects, from the author's comparisons between quite small arches of definite shape.

The diagram, Fig. 19, illustrates the very carefully made comparisons between a three-hinged and a two-hinged arch with elastic tie rod for the Hacker Bridge in Munich.† In the right half the theoretically determined cross-sections for a three-hinged arch are given, and in the left half those for a two-hinged arch, in square decimeters. Pure tensile strains are indicated by +; pure compressive strains by —, and members subject to both kinds of strain by [.

The result of this comparison is very unfavorable to the three-hinged arch, even when only the theoretically determined volumes are considered, inasmuch as it requires 11.3% more material than the

* Engesser: "Supplementary Forces and Secondary Strains in Iron Truss-Bridges," 1892 and 1893.

† *Zeitschrift des Vereins Deutscher Ingenieure*, 1893. "Construction of the Hacker Bridge," p. 1441

Mr. Dietz. two-hinged arch, and this difference becomes still greater when the additional material required for the center hinge is taken into account. A still greater disadvantage for the three-hinged arch appears when its deflection is considered, as it amounts to 30.5% more than that of the two-hinged; it was calculated carefully for both arches, according to the well-known method by Mohr, taking into account the change of length in every member.

The writer maintains that the question, whether under all circumstances the statically determinate framework is superior to the indeterminate, cannot at all be answered on the basis of a one-sided comparison, considering only the theoretically required amount of material. It can only be determined in each case by weighing carefully the influences of all the factors, and requires, not only thorough theoretical knowledge, but also, a large amount of comparative material and experience.

Mr. Sohn, Prof. JOSEPH SOHN (by letter).—The exact design of statically indeterminate frameworks, as explained in the paper, is based essentially on the idea that the stresses of the superfluous bars under a certain loading are not computed but assumed. In the case of a single given loading (constant loading), the author assumes arbitrarily the stresses in the superfluous bars, deduces thence and from the loading the stresses in the remaining (necessary) bars and designs on this basis the cross-sections of the various members. The equations of elasticity determine then the differences between the figure length and the actual unstrained length of each superfluous bar. Thereby, the construction is designed, and in practice it only remains to give to the bars the corresponding length. The more nearly this condition is attained the more nearly the actual stresses approach the designed.

If a statically indeterminate framework is subject to the several different loadings (variable loading), the author assumes the stresses in the superfluous bars under one of the given loadings or, more generally, he sets an equal number of further conditions; in the equations of elasticity, each relating to the difference between the stresses in the bars under two of the given loadings, and therefore not containing the differences between the figure and actual unstrained length of the superfluous bars, the author expresses the section areas of every bar by the corresponding stress under the most unfavorable loading; the equations of elasticity determine, then, the stresses in the superfluous bars under each of the given loadings; thence result the stresses in the necessary bars, and it remains to compute the differences between the figure length and actual unstrained length of the superfluous bars.

The case of a constant loading offers, in designing, no difficulty; with variable loading, on the contrary, the design, even under the simplest conditions, is exceedingly complicated and laborious. The

author himself accents it expressly and adds force principally upon the proof that statically indeterminate systems are in all respects (economy, stiffness, safety) inferior to the determinate constructions, and that only these last systems may be recommended in practice. Mr. Sohn.

Although there are cases where it will not do to exclude statically indeterminate forms, the writer agrees with the author that statically determinate systems generally are to be preferred, particularly for the reason that slight imperfections of the actual construction, unavoidable in practice, do not influence essentially the stresses in the bars. In statically indeterminate constructions that influence is, on the contrary, very considerable; wherefore important divergences between the computed and actual intensities of stress are possible and probable. A simple example, given by the author, shows evidently how much the intensity of stress can be altered by the incorrect length of a bar.

G. Jung,* Esq. (by letter†).—The writer has read Mr. Cilley's paper on indeterminate frameworks with much interest and takes pleasure in finding that he entirely agrees with him. Mr. Jung.

For a long time the writer has maintained that statically determinate frameworks are preferable to indeterminate ones. His reasons for this opinion being about the same as those which the author has so well set forth, he considers it useless to repeat them here.

On the other hand, he has for many years, in his course on graphical statics at the Superior Technical Institute of Milan, called the attention of students to the paper of Maurice Levy (mentioned also by Mr. Cilley), and has invited them to study the argument treated therein. With considerations of a general character, analogous to those given on pages 570 and 571 of the paper, the writer emphasizes the simplicity of the calculation and of the hypotheses required by statically determinate frameworks as compared with the complicated calculations and the necessity of further hypotheses, required by indeterminate frameworks, and concludes with pointing out the superiority of the former structures over the latter, at least from the theoretical and purely statical (and economical) point of view.

As for the dynamical phenomena, the writer does not know of any; but the author's observations on page 572‡ seem to be just. The writer does not know, either, what the good reasons are which could make one prefer indeterminate rather than statically determinate frameworks, which latter have in their favor, besides, the practice and the large experience of American engineers.

Frank H. Cilley, S. B. (by letter).—The writer believes that Mr. Goldmark's objections rest, in part, on a misapprehension. The use of methods of calculation based on the principle of virtual "Velocities" or "Displacements" or "Work," as it is variously termed, or Mr. Cilley.

* Associate Manager of the *Annali di Matematica*, Milan, Italy.
† Translated.
‡ *Proceedings*, Vol. xxv.

Mr. Cilley. which result from the principle of least work, which is simply an integral form of the preceding, is the simplest as well as the most correct means of determining changes in stress and strain in indeterminate structures. Where such structures exist, and, therefore, must be examined, the writer most strongly approves the use of these methods. And in designing, if engineers will use indeterminate forms, these methods are still to be preferred. But in this case the present tentative method of application must be used, as the direct and exact method outlined by Mr. Goldmark is too difficult to be practicable. The writer's point is that while the methods just referred to are the best known, still, being indirect and tentative, and embodying no principle that guides to good rather than bad proportions, even their use in designing is necessarily unsatisfactory, and, consequently, designs depending on their use will be correspondingly defective. The writer's objections are directed rather to the unnecessary use of systems of construction involving these uncertain, wearisome and unsatisfactory calculations than to the methods of making these calculations.

Regarding the alleged tendency to multiple systems of webbing, the writer would refer Mr. Goldmark to Professor Ritter's statement in his contribution to this discussion, that "the use of the multiple intersection system has commonly been dropped" as a result of German investigations of the subject.

Mr. Goldmark's references to the bibliography of the subject are valuable. The writer personally prefers the use of the principle of "Virtual Velocities" (displacements or work), so clearly set forth in Professor Swain's article, "On the Application of the Principle of Virtual Velocities to the Determination of the Deflection and Stresses in Frames," elsewhere referred to, to the use of the integral principle of "Least Work," of which an excellent exposition by Professor William Cain will be found in the *Transactions** of this Society. The former principle, while leading to precisely the same equations as the latter, is much more objective, and, in the writer's opinion, is less likely to introduce errors.

Mr. Lindenthal's objections raise many most interesting points which the writer only fears his present very limited time will not permit him to discuss with the thoroughness which is desirable. He trusts that a further opportunity will permit of his correcting present shortcomings, and that brevity will be pardoned in this instance.

One of Mr. Lindenthal's first objections is to the comparison of a given indeterminate framework with a determinate framework of included figure, a procedure which he characterizes as unfair. In this the writer will agree with him, except that the unfairness is not to the indeterminate, but to the determinate form. For the indeterminate

* *Transactions*, Am. Soc. C. E., Vol. xxiv, p. 265.

form is any given form, and, certainly, it would be fair in theory and always permissible in practice to use in its place another framework of included figure. It rises no higher, it descends no lower, it extends in nowise beyond the bounds of the given framework, and offers at least as great clearances in all directions. What just objection, then, can be made to its substitution for the given framework? And if it is true that this leads to a greater "effective" height or depth in the substituted framework, that is only one of the legitimate advantages of the substitution. It is in part because of such advantages that the determinate forms are gainers over the indeterminate. But the comparison, even so, is unfair to the determinate framework, for all the proportions of the indeterminate framework may be those which most favor it, while the determinate framework, whose figure is formed simply by dropping out certain lines of the figure of the given indeterminate framework, without any alteration in proportions, may not, and very likely will not, be the best determinate framework whose figure simply does not exceed the bounds of the figure of the given indeterminate framework. This latter would certainly be a fairer basis, but even that would not be wholly fair. Usually, bounds in many places exceeding those of the given indeterminate framework would be equally permissible. It would only be fair to permit the search for a superior determinate framework to extend to any within these bounds. It must be remembered that the indeterminate framework is supposed to be given and the question raised is: Can a determinate framework, which would satisfy equally well the actual limiting conditions, be found, which would advantageously replace the given framework? Could this be shown in general to have an affirmative answer, the superiority of determinate over indeterminate construction would thereby be demonstrated conclusively. Mr. Cilley.

But it is not possible to thus broaden the limits in a general discussion. We must remain within limits which are beyond question, and those surely are the boundaries of the given indeterminate framework. As a result, while the demonstration that there exists a superior determinate framework under these conditions is absolutely conclusive, the demonstration of the contrary would still leave open the possibility of a reversal through fairer conditions.

Mr. Lindenthal's next point is an objection directed to the very considerable proportion of depth to span in the arch of the second illustration. It is one-eighth, whereas in practice one-sixteenth is about the maximum and one thirty-second more frequent. Now, this arch was proportioned as a two-hinged and not as a three-hinged arch. The small number of its members (required to reduce the work of an "exact design") alone is responsible for this considerable depth. The three-hinged design obtained by dropping out the middle bar of the two-hinged design is far from being of a superior character.

Mr. Cilley. For the limited number of members used, the two-hinged design is by no means favored in its proportions. If it gains decidedly in effective arch rise at the center through this great depth of the two-hinged arch, it may be contended, on the other hand, that the great depth of the two-hinged arch at the middle should secure it great rigidity. We should expect that where the characteristic difference of two types was most marked, there we would find the most marked expression of the superiority of one or the other. And surely it is the absence of all stiffness at the center of the three-hinged arch, and the considerable stiffness at the center of the two-hinged arch, which are the most characteristic contrasting features of the two. If, where these are most marked, we find economic and other differences small, how much less will they become as these differences in character become less marked. It must be remembered that the writer is contending that decided advantages in economy, stiffness and safety are not obtainable through the use of indeterminate frameworks, and, therefore, that other considerations favoring determinate construction decidedly outweigh any small advantages in these directions which indeterminate structures may occasionally possess. It is, therefore, a very important point to have shown, as in the illustration to the paper, that even in an extreme case, where large divergence was to be expected, actually, it was small. Therein, in part, is the justification for the statement that in actual and usual cases the differences will be much smaller still.

We now come to another point, and this is a very important one, in the comparison of determinate and indeterminate forms—the question of flexibility. If a structure be very flexible, that is, permit of relatively very considerable changes of form, if it is an indeterminate structure, it is evident that the importance of adverse factors, such as temperature changes, yielding supports and inaccurate construction, will be greatly lessened. That is to say, in slender structures the defects of indetermination tend to become negligible. Now, an arch with a rib depth less than one thirty-second of the span certainly is a slender and relatively flexible structure, and, if arches are usually of such proportions as Mr. Lindenthal states, then they are usually in the category of structures in which the evil consequences of indetermination are minimized. Ordinary, stiffened suspension bridges are also in this category.

Now, these flexible types really form a class by themselves. Their distortions under load are so considerable that the ordinary theory of rigid structures does not closely apply to them. The terms "statically determinate" and "indeterminate" cease to have their old meanings, and consideration of elastic displacements becomes necessary in all cases. It is true that such consideration is not ordinarily given, but that is chiefly because of our ignorance and its difficulty—not because it is not needed.

The writer's study can make no pretense of including such structures. Based on the application of the principle of virtual displacements, it certainly does not apply when the actual displacements result in very sensible directional changes. And it will be well, at this point, to eliminate from this discussion the discussion of the comparative merits of determination and indetermination in such structures. It is an interesting subject, but occupies a field by itself. Here, we are only concerned with the location of its border line. The writer ventures the temporary suggestion that structures whose changes in form under load result in a change of more than 10% from the stresses which would result if the changes in form were insignificant, be considered as of the class of flexible structures here excluded. Mr. Cilley.

Limiting ourselves, then, to the consideration of non-flexible structures, the writer's statement of the great importance of temperature stresses in indeterminate structures is well founded, and his illustration with the deep arch a proper example. The suspicion is probably justified that, in indeterminate arches in which the temperature stresses have been found to be very small, the distortions are actually very considerable, and the calculated stresses under load, obtained in the usual ways, far from expressing the actual stresses.

Mr. Lindenthal tells us that:

"Comparisons of carefully worked out designs of different arch types, made by him and others, have convinced him that, other things being equal, the indeterminate types are invariably more economical."

Without questioning his sincerity in this statement, it may fairly be asked that he shall produce the data in order that judgment may be passed on the fairness of the comparisons. The writer's experience in these matters is, that really fair comparisons are practically unknown, not because of prejudice on the part of those making the comparisons, but because of the unconscious introduction of unreasonable limitations. Thus, if a three-hinged arch is to be designed for comparison with a two-hinged arch, the middle hinge is usually placed midway between the top and bottom chords, or at the bottom chord, in spite of the obvious fact that the horizonal thrust is diminished by putting the hinge as high as possible; that is to say, in the top chord. Or, if it is a comparison of the three-hinged arch with a hingeless arch, two of the hinges are always placed at the abutments, in spite of the fact that it may be decidedly more advantageous to put them farther out, and that it is legitimate to do this and subject the abutments to a bending moment which they would have to resist in any event with the hingeless arch.

Another form of unfair discrimination is that mentioned by Professor Jacoby,* where he speaks of the designing of a three-hinged

* *Proceedings*, Am. Soc. C. E., Vol. xxv, p. 717, or *Transactions*, Am. Soc. C. E., Vol. xliii, p. 31.

Mr. Cilley. arch to compare with a given two-hinged arch, and giving the sections of the lower chord of the former such proportions that, in spite of its being heavier, its radius of gyration averaged 4% less than that of the sections of the lower chord of the two-hinged arch. In spite of this fact, however, in this case the three-hinged design proved slightly the more economical. The designing of the cross-sections is a most important factor in fair comparisons, and it alone, by its variation, may easily throw the balance in favor of one type or the other in the comparison of actual designs. This fact is one of those which render the comparison of actual designs so unreliable a basis for determining the relative economy of types.

We now come to a point on which Mr. Lindenthal correctly insists, but, the writer fears, without due regard to proportion—the fact that changes in temperature modify the stresses, even in many determinate frameworks, contrary to the prevalent idea. But Mr. Lindenthal goes so far as to state that:

"Three-hinged arches are not only not free from temperature strains, but for certain conditions such strains are just as large as in arches of the two-hinged type. * * * The supposed advantage in that respect of the three-hinged over the two-hinged arch is illusory."

While the changes in stress resulting from temperature (which are not precisely temperature stresses) are facts and should be considered, the writer feels that Mr. Lindenthal's quantitative statements should be accompanied by the data on which they are based so that we may see for what "certain" conditions these stresses accompanying temperature changes are so considerable.

If we look into these so-called temperature stresses of Mr. Lindenthal a little closer we shall note that they are simply a special case of the modification of the stresses in a structure resulting from changes in its form, however brought about. The loadings bring about such changes in form, and, as previously noted, to such an extent in the more flexible structures that the ordinary theory which neglects such changes is seriously in error. Now the fact is simply this, that structures whose figures are altered seriously by temperature changes are likely to have their figures altered even more seriously by their loading; that is to say, are to be classed among the flexible structures mentioned previously. While calling attention to these temperature changes Mr. Lindenthal is neglecting other, even more important, changes which occur in these cases.

And another fact is to be observed: These neglected changes, whether from temperature or load, occur in the indeterminate frameworks as well as in the determinate, and add their effects to one as well as to the other; so that Mr. Lindenthal's neglected factor is one that must be applied, both to the indeterminate and the determinate frameworks. Very flat arches, whether three-hinged, two-hinged or hingeless, suffer serious alteration in their stresses from those ordinarily determined,

through their flattening very sensibly further, whether from fall of temperature or from the application of the loads. It is a mistake to suppose that the present usual theory of indeterminate structures takes this flattening into account, whether in connection with temperature or load stresses. Actually, it neglects it as completely as does the ordinary theory of determinate structures. To see this and to enable us to form a better idea of these alterations of stress, we must develop their theory. Mr. Cilley.

Theory of Changes of Stress in Frameworks Due to Changes in Form.

Suppose we denote by P the loads, by S the stresses, by R the reactions, and by α the angles of the bars, loads and reactions with (say) the horizontal, confining ourselves here, for simplicity, to plane frameworks. Then, as we know, we have, on the ordinary supposition of no appreciable change in form of the framework under load, equations for each joint, of the form :

$$\left.\begin{aligned}\Sigma P \cos.\ \alpha + \Sigma R \cos.\ \alpha + \Sigma S \cos.\ \alpha = 0\\ \Sigma P \sin.\ \alpha + \Sigma R \sin.\ \alpha + \Sigma S \sin.\ \alpha = 0\end{aligned}\right\}\ldots\ldots\ldots(A)$$

But if the framework actually does change in form and the α become $\alpha + \varDelta_\alpha$, the S become $S + \varDelta_S$, the R become $R + \varDelta_R$, and the P become $P + \varDelta_P$, then our joint equations become :

$$\left.\begin{aligned}\Sigma (P + \varDelta_P) \cos.\ (\alpha + \varDelta_\alpha) + \Sigma (R + \varDelta_R) \cos.\ (\alpha + \varDelta_\alpha) +\\ \Sigma (S + \varDelta_S) \cos.\ (\alpha + \varDelta_\alpha) = 0.\\ \Sigma (P + \varDelta\ ) \sin.\ (\alpha + \varDelta_\alpha) + \Sigma (R + \varDelta_R) \sin.\ (\alpha + \varDelta_\alpha) +\\ \Sigma (S + \varDelta_S) \sin.\ (\alpha + \varDelta_\alpha) = 0.\end{aligned}\right\}\ldots(B)$$

Each of these terms, as $(S + \varDelta_S) \cos.\ (\alpha + \varDelta_\alpha)$ may be expanded as $(S + \varDelta_S) \cos.\ (\alpha + \varDelta_\alpha) = S \cos.\ \alpha \cos.\ \varDelta_\alpha - S \sin.\ \alpha \sin.\ \varDelta_\alpha + \varDelta_S \cos.\ \alpha \cos.\ \varDelta_\alpha - \varDelta_S \sin.\ \alpha \sin.\ \varDelta_\alpha$.

Now suppose the $\varDelta$ all relatively small, then we could write $(S + \varDelta_S) \cos.\ (\alpha + \varDelta_\alpha) = S \cos.\ \alpha - \varDelta_\alpha S \sin.\ \alpha + \varDelta_S \cos.\ \alpha$, and similarly with the other terms of (B), and taking Equations (A) into consideration we thus obtain from Equations (B) the difference equations for each joint:

$$\left.\begin{aligned}\Sigma \varDelta_P \cos.\ \alpha + \Sigma \varDelta_R \cos.\ \alpha + \Sigma \varDelta_S \cos.\ \alpha - \Sigma \varDelta_\alpha P \sin.\ \alpha\\ - \Sigma \varDelta_\alpha R \sin.\ \alpha - \Sigma \varDelta_\alpha S \sin.\ \alpha = 0\\ \Sigma \varDelta_P \sin.\ \alpha + \Sigma \varDelta_R \sin.\ \alpha + \Sigma \varDelta_S \sin.\ \alpha + \Sigma \varDelta_\alpha P \cos.\ \alpha\\ + \Sigma \varDelta_\alpha R \cos.\ \alpha + \Sigma \varDelta_\alpha S \cos.\ \alpha = 0\end{aligned}\right\}\ldots(C)$$

If the framework be determinate, then, since the P and α are known, the R and S determinable from the Equations (A) or their equivalents,

Mr Cilley. the Δ_a (except for the reactions) very approximately determinable from the elastic displacements of the joints, for the given materials and sections, due to the stresses S, temperature and other causes, and the Δ_P either zero or definitely related to these displacements, Equations (C) suffice to determine our remaining unknowns, that is to say, the quantities Δ_R, Δ_{a_R}, and Δ_S.

The above result suffices for the solution of the problem, but it is capable of an exceedingly simple and objective interpretation which extends at once to the calculation of these changes in stress in all the methods and resources, graphical and analytical, already developed for the calculation of the ordinary stresses. Consider the following:

If we inspect Equations (C) and compare them with Equations (A) we perceive that they are simply the equations of equilibrium at the joints of the given framework subject to the loads formed by taking the changes in the original loads, together with the further loads $\Delta_a S$ at right angles to the corresponding bars. The resulting stresses in the framework under these loads are the desired changes in stress. So we have only to find this differential loading, which is readily done when the displacements of the joints have been determined (as by a Williot diagram), and then we may proceed with the solution of the stresses by any of the usual methods, graphical or analytical, obtaining thus the change in stress resulting from change in form of the framework, in the case of any determinate framework.

Applying the foregoing to the three-hinged arch of the second illustration of the paper, to determine the changes in stress due to change in figure resulting from a fall in temperature, we have the following: For a fall in temperature of 100° Fahr., if the arch did not alter its form, we would have a reduction of span of 0.4 × 0.065 ft. (see page 616*) and a consequent descent of the middle hinge of $\frac{12}{40}$ of this, or 0.12 × 0.065 ft. But we suppose the span actually remains constant, and therefore the middle hinge must sink further,

$$0.2 \times 0.065 \times \frac{20}{12} = 0.333 \times 0.065 \text{ ft.}$$

The total sinking of the middle joint will therefore be $(0.333 + 0.120) \times 0.065$ ft. $= 0.030$ ft., nearly. That is, all bars of the left half will rotate through the common angle,

$$\tan.^{-1} \frac{0.030 \times 20}{20^2 + 12^2} = \tan.^{-1} \frac{0.6}{544} = \tan.^{-1}\, 0.001103 \text{ (about } 3\tfrac{3}{4} \text{ minutes)},$$

in a right-handed direction, and all bars of the right half will rotate through the same angle in a left-handed direction.

Consider, first, the consequent changes in the stresses under a full loading of P on each joint (loading L, C, R).

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Mr. Cilley.

We have in Fig. 15*a* the differential loading for this case and the resulting stresses which are the changes in stress sought. The value of Δ (R cos. α), or the change in the horizontal thrust, not given in Fig. 15*a*, is 0.00444 P, which permits of ready verification. Next consider the loading C R (P on center and right joints) under which bars b, c and e (see Fig. 2) have their greatest stresses.

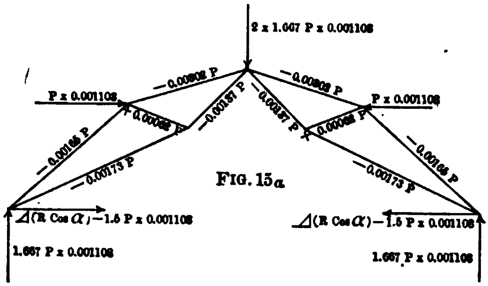

FIG. 15*a*

We have, in Fig. 15*b*, the corresponding differential loading and the resulting stresses in the bars b, c and e, which are the changes in stress in these bars sought. The change in the horizontal component of the reaction in this case is found to be Δ (R cos. α) $= 0.003125$ P, which may easily be verified otherwise.

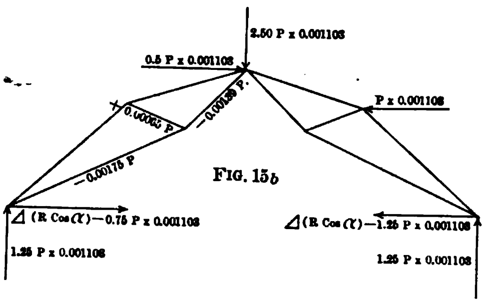

FIG. 15*b*

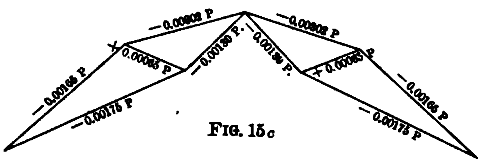

FIG. 15*c*

Putting down the changes in stress in the bars a and d under the loading L C R, and in b, c and e under the loading C R (and L C) we have, in Fig. 15*c*, the additions to the maximum stresses to be made on account of a change in temperature of 100° Fahr. Compare this with Fig. 15. Taking a change in temperature of only 50° Fahr. (that is, a range of temperature of 100° Fahr.), as in the case of the two-hinged arch, we find our additions are (compare Fig. 10) 0.09% in bar e, as against 22% for the two-hinged arch; 0.09% in bar c as against 20%; 0.09% in bar b as against 10%; 0.09% in bar a as against 11%; 0.035% in bar d as against 6%, to say nothing of no stress here to compare with the 45% temperature stress in bar z of the two-hinged arch.

This comparison, showing the stresses in the case of the three-hinged

Mr. Cilley. arch resulting from temperature changes to be only about one-two hundredth of the ordinary temperature stresses of the two-hinged arch, will be found a sufficient answer to Mr. Lindenthal's statement:

"If the author will trace the strains, due to the falling and rising of the center hinge from changes of temperature, he will find them by no means of negligible smallness, even for the large ratio of rise to span in his inconclusive figure."

But this is not all. The "temperature" (?) stresses we have just traced out in the three-hinged arch also occur in the two-hinged arch, in addition to those already calculated. Let us now develop the theory of these changes.

The theory of the changes of stress in indeterminate frameworks due to changes in form is not quite so simple as that for the determinate frameworks just set forth, but is based on the latter. At a first glance it would appear that one might simply find the differential loading for the indeterminate framework, and thence obtain the changes in stress sought simply as the stresses in the given indeterminate framework under this differential loading, on the supposition of no stress under no loading. But this is not quite correct as the following analysis will show.

We have seen (page 557*), that the difference of the figure length L_i from the actual length l_i of a superfluous bar is expressed by the equation

$$\sum_{f=1}^{f=m} \frac{S_f'^{(i)} S_f l_f}{E_f A_f} = L_i - l_i \dots\dots\dots\dots\dots (D)$$

provided only negligible changes in direction of the bars occur. If, as a result of the actual changes in direction of the bars, the stresses actually become $S_f + \Delta_{S_f}$ and the coefficients $S_f'^{(i)}$ become $S_f'^{(i)} + \Delta_{S'_f{}^{(i)}}$ then we must have

$$\sum_{f=1}^{f=m} \frac{(S_f'^{(i)} + \frac{1}{2}\Delta_{S'_f{}^{(i)}})(S_f + \Delta_{S_f})\, l_f}{E_f A_f} = L_i - l_i \dots\dots\dots (E)$$

And supposing, as before, that the Δ are small, we readily derive the simple difference equations

$$\sum_{f=1}^{f=m} \frac{S_f'^{(i)} \Delta_{S_f} l_f}{E_f A_f} + \frac{1}{2}\sum_{f=1}^{f=m} \frac{\Delta_{S'_f{}^{(i)}} S_f l_f}{E_f A_f} = 0 \dots\dots\dots\dots (F)$$

These Equations (F) together with Equation (C) actually determine the changes in stress, Δ_{S_f}. The second term on the left-hand side of Equations (F), which would have to be zero if the Δ_{S_f} were simply the stresses for the differential loading on the supposition of no stress under no load, shows the error that supposition would involve.

Instead of the purely analytic procedure of combining Equations (F) and (C), the following procedure is likely to prove preferable.

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The stresses in the non-superfluous bars are expressed in terms of those in the superfluous bars by the equations (see p. 582*). Mr. Cilley.

$$S_f = S_{f_0} + \sum_{i=h}^{i=m} S_i S_f'^{(i)} \dots\dots\dots\dots\dots\dots (G)$$

on the supposition of no change in form. Actually, we must have

$$S_f + \Delta_{S_f} = S_{f_0} + \Delta_{S_{f_0}} + \sum_{i=h}^{i=m} (S_i + \Delta_S \;) \; (S_f'^{(i)} + \Delta_{S'_{f}(i)}) .. (H)$$

and again, supposing the Δ small, these equations give the difference equations

$$\Delta_{S_f} = \Delta_{S_{f_0}} + \sum_{i=h}^{i=m} \Delta_{S_i} \; S_f'^{(i)} + \sum_{i=h}^{i=m} S_i \, \Delta_{S'_f(i)} \dots\dots (K)$$

Now these Equations (K), together with Equations (F), may form the basis of our solution. In them the Δ_{S_f} are the quantities sought. The $\Delta_{S_{f_0}}$ being the changes in stress of the framework composed of the non-superfluous bars, under the given loading, due to the change of its form for the joint displacements of the given indeterminate framework under that loading; and the $\Delta_{S'_f(i)}$ being the corresponding changes in the coefficients $S_f'^{(i)}$. These are both determinable by the method for determinate frameworks, explained previously.

As to the solution of Equations (F) and (K), besides the analytic methods, we may readily apply to them any of the usual graphic methods for the determination of stresses in indeterminate frameworks. The similarity of their forms to those of the equations for stresses in indeterminate frameworks will suggest the manner of utilizing the same methods. Thus one may take advantage here, as in the case of the calculation of changes in stress accompanying changes of form in determinate frameworks, of all the accumulated methods and resources, graphical and analytical, now in our hands for the calculation of the ordinary stresses on the supposition of no change in form.

The writer regrets that his present limited time prevents his illustrating the use of these methods by a calculation of the additional stresses in the two-hinged arch of the paper, resulting from the flattening of that arch accompanying a fall of temperature. It is evident, however, that they would be quantities of the same proportions as those already found for the three-hinged arch.

In concluding this brief exposition of the theory of the calculation of changes in stress in frameworks accompanying small changes in form, the writer believes he can commend its use for cases where the actual changes are really quite considerable, that is to say, for the approximate calculation of the corrections to be given for the stresses in flexible frameworks, as slender arches and stiffened suspension bridges.

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Mr. Cilley. Although the work is laborious, it is only laborious in the same way and degree as the work of the usual methods for the ordinary calculations on which it is based, and it certainly is far simpler than the application of higher mathematics called for by the present rudiments of methods, proposed or used in such cases. The somewhat sketchy character of the writer's present exposition he trusts will be pardoned as the treatment of the subject here given is wholly new, so far as the writer knows, and was developed, at very brief notice, by him, for this discussion.

Before leaving the subject of temperature stresses the writer desires to note his exception to Mr. Lindenthal's quantitative statement regarding the extra material required to meet the three-hinged arch temperature stresses in the cases of the Washington arch and of the Niagara Falls road bridge (or Clifton) arch. Such claims are too easily made to be acceptable when unaccompanied by any figures in their support, and, above all, when rendered doubtful by general analysis, as in the present case.

To Mr. Lindenthal's claim, that "a proper comparison" of the strain sheets "of the two types of arch treated by Mr. Cilley" (two and three-hinged), demands a correction "for the large bending moments resulting from the immobility of the hinges, incorrectly ignored by him," the writer feels that he must strongly take exception. Mr. Lindenthal forgets that in the paper it was particularly specified that only ideal frameworks with frictionless joints were under consideration. Moreover, the pin is not a necessary connection member for this purpose. Thin plates have not only been proposed, but also used. In the Kaisersteg bridge* at Oberschoenweide, near Berlin, designed by Müller-Breslau, such a plate hinge was used, and in France an entire set of joints has been constructed recently with such plates, in a bridge of 132-ft. span, described in the *Annales des Ponts et Chaussées*.

Thus, while a correction is necessary where pins are used, they are really unnecessary indeterminate adjuncts, and therefore are not properly to be considered in such a comparison as that made in the paper.

Mr. Lindenthal says that temperature stresses and hinge friction require additions to the sections of both two and three-hinged arches, which cannot be ignored, and follows this with the somewhat extreme statement—"When made (these additions), the two-hinged arch will in every case show great economy over the three-hinged arch." Again, the writer feels that we are entitled to the figures in proof of this bare assertion. The analytic considerations, thus far exposed, by no means support it.

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* Described in *The Engineering Record* of Feb. 17, 1900.

Mr. Lindenthal holds that it "is not necessary, either for safety or economy," that the strains in an indeterminate framework "be determined with the same exactness as in statically determined structures." If Mr. Lindenthal thus unfairly favors indeterminate designs, it may well be that he will find in them the higher economy he claims, but it is certainly competent for the opponents of such designs to take exceptions to such procedure. It must be remembered that it does not take much favoring to eat up the small margin of a few per cent. which will ordinarily limit the difference in material required in the two cases. The smallness of this difference is precisely the reason why the writer argues rather the lack of economic advantage in the use of indeterminate forms, than its presence in the use of the determinate forms. Mr. Cilley.

Mr. Lindenthal states that "no failures of bridges, caused by their indeterminateness, have yet occurred." Has he not, perhaps, overlooked some cases? Partial failures due to indetermination have been anything but wanting. Many a weak member has been broken or has broken its connection when carried along by the movement of some heavy member to which it was improperly attached; many a case of initial strain, permitted by the use of redundant members, has resulted in rupture; many a continuous floor system has failed in the rivets, under the stresses actually coming upon it, for which it was not designed; and in accidents like the buckling of the lower chords of the stiffening trusses of the Brooklyn bridge over the towers, we have illustrations of the consequences of indetermination in the case of more important parts. It may be said that these were due, not to the use of indeterminate construction, but to faulty calculation, or neglect to calculate at all, or to faulty adjustment. But these things arose purely from the indetermination and may therefore properly be charged against it. Far from indetermination never being a cause of failure, it has been a fruitful cause, especially in other structures than bridges, where temperature strains have worked havoc.

We now come to the last of Mr. Lindenthal's objections, which is to deflection as a criterion of rigidity. He says: "What is a rigid bridge? If we take rigidity in metallic bridges to mean absence of vibration, then deflections are a deceptive criterion."

If this were merely a question of difference of definition of the word "rigidity," it would have no further importance, but Mr. Lindenthal continues: "The public considers a bridge which vibrates, as an inferior structure; and one which does not, as a superior one. This ought to be the guide, also, to the engineer." But this argument of "*Vox populi vox Dei*" is hardly convincing, and Mr. Lindenthal's illustrations only serve to increase the writer's distrust.

Vibration, far from being an indication of anything defective, is the evidence of a highly elastic and rigid constitution. Only rigid and elastic large bodies vibrate rapidly, and therefore, in a fashion

Mr. Cilley. very noticeable to us. The vibrations of a slack rope are not marked, but it is scarcely rigid. A Peruvian rope suspension bridge is not notable for its vibration, but even the public would scarcely regard it as superior. Many a loose-jointed wooden structure is free from vibration, while in similar metal structures it is most marked, but the wooden structure is hardly therefore the superior one.

Wherever great rigidity, elasticity and lightness go together, vibration is likely to be noticeable. Where, on the contrary, there is loose or slack construction, imperfect elasticity and great mass, vibration will be imperceptible. The writer can scarcely believe that Mr. Lindenthal had seriously considered the bases of vibration before he thus proposed it as the criterion for distinguishing bad from good construction. Large deflections, which mean flexible construction, really are objectionable, because they interfere seriously with high speeds, and, moreover, allow of dangerous swaying in high winds. But rapid vibration is an evil affecting the imagination far more than it affects the structures. Moreover, Mr. Lindenthal's illustrations show that marked vibration occurs equally in indeterminate and determinate structures. It is, in fact, a matter largely independent of the presence or absence of superfluous bars.

In concluding this part of his reply, the writer feels justified in protesting at the statements wholly unsupported by evidence, and some of which analysis has shown to be incorrect, which characterize this part of the discussion. It was his hope that those who made counter claims to the arguments in the paper would adduce their evidence in such form as to permit of its independent consideration. In the absence of such evidence it can scarcely be expected that these claims will receive much consideration, and, certainly, we cannot be expected to accept them.

In replying to Professor Ritter's comments, the writer desires to express at the outset his appreciation of the very fair and reasonable position taken by him, as well as of the entire absence of any misapprehension or misconstruction of the paper in these comments. In the hands of men as able and as honest as Professor Ritter, the use of indeterminate frameworks loses many of its objections. But, since few engineers favoring indeterminate construction are equally well informed and fewer are as fair, its use is generally subject to the full force of the writer's objections.

Some of the points touched on by Professor Ritter have been considered by the writer in his reply to Mr. Lindenthal, but there remain several points which it is desirable to consider here.

The shorter unsupported lengths of the struts is an advantage which has frequently been cited in connection with lattice webbing, and as a justification of the European custom of riveting members together wherever they may cross. But it seems to be questionable

whether the apparent advantage thus obtained is not illusory. As was long ago pointed out by Maurice Levy, the use of multiple systems of webbing results in a reduction of the sections of the posts which may well much more than offset the reduction of free length due to attachment at points of intersection. And this is apart from the fact that the free lengths for side deflections (deflections out of the plane of the truss) are not reduced through this arrangement in anything like the degree that the free lengths for deflections in the truss planes are reduced. But this is not all. Such attachment of points of intersection results in bending moments (when it does not result in anything more), which are rarely if ever considered, although sometimes very large. When these are taken into consideration it will be found that the consequent added secondary stresses much more than offset any advantage from shortened free length. Moreover, this practice of riveting every member to every other member wherever there is a chance becomes a wholly indiscriminate and blind action which sometimes entails serious unforeseen results, and has been responsible for no small part of the failures in indeterminate construction. The fact that not more than two passing members can be connected at the same point without entailing, in addition to bending stresses, direct axial stresses, is usually overlooked or not appreciated. Mr. Cilley.

The writer's view is, that as far as possible we should design only as we can and do calculate and understand, and that the procedure he is now criticising, the consequences of which are neither calculated nor understood, should be put aside by all who prefer the guidance of their intelligence to slavish copying of ill-founded methods of the past. Some day it will be recognized that every unnecessary connection is only an added restraint upon the free exercise of function, and that, as the writer has elsewhere pointed out, "Statical indetermination in a structure is always to be regarded as self-interference with efficiency."

In regard to the objections to the expense of hinges, constructed as at present, with pins, the writer ventures to suggest that this difficulty, as well as that of the shocks at the hinges, mentioned by Professor Ritter, may be overcome by the substitution of thin plates for pins. Flexible joints by no means depend on the use of pins for their realization, as is so commonly assumed.

Regarding discrepancies of length of members, the writer would point out the consequences in this direction which the theory of probability tells us we must expect. We know that an error ε in the length of any member of a statically determined framework, will change the distance, between any two of its joints not directly connected, by the amount $S'\varepsilon$ (where S' is the stress in the member due to a pair of equal and opposite forces of unity applied each at one of the two joints). Now suppose each bar f has a corresponding probable

Mr. Cilley. error ε_f in its length (which error may equally well be + or —), then the probable resultant error Δ, in the distance between the two joints, as a consequence of all these probable single errors ε_f, we know from the theory of probability will be

$$\Delta = \sqrt{\Sigma\,(S'\varepsilon)^2}$$

Let us apply this to an example:

Suppose we have a two-hinged arch of, say, 500 ft. span, and 90 ft. rise, with parallel chords 15 ft. apart, and that it is built out from the abutments to meet at the center. Suppose that we have forty panels and that the probable error of each member is only $\frac{1}{64}$ in. This is moderate for even the best of work, that is to say, with all lengths determined by milled surfaces brought into contact for riveted joints, or with pin bearings, as very careful, repeated measurements made by the writer with an excellent steel tape, firmly clamped at one end and subject to a uniform pull by means of a spring balance at the other end, have convinced him. For simple riveted connections the work would be sensibly less accurate. Here it will readily be seen that the average value of S' for the chord members is about two-thirds, while that for the web members is less than one-fifteenth, but more than one-twentieth. There will be 80 chord members, which alone would cause a probable error in the closing middle chord bar of

$$\frac{2}{3}\sqrt{80} \times \frac{1}{64} = \frac{3}{32} \text{ in., about.}$$

With a simple triangular system, as we will suppose, there would also be about 80 web members, and taking for them the smaller average value of S' as one-twentieth, the probable error in closing of the middle chord bar due to their errors would be

$$\frac{1}{20}\sqrt{80} \times \frac{1}{64} = \frac{1}{128} \text{ in., about.}$$

The resultant probable error for both chords and webbing would be

$$\sqrt{\left(\frac{3}{32}\right)^2 + \left(\frac{1}{128}\right)^2}$$

(since chord and web members are equally numerous) or practically $\frac{3}{32}$ in., as for the chords above. This gives an idea of what may be expected in the final result from a given degree of accuracy of workmanship, and so tells us what degree of accuracy it is necessary to maintain in a given case. Some such method should be used where indeterminate frameworks are constructed.

Professor Ritter states that "In general, structures with hinges (arches or cantilevers) are less stiff than those without hinges." The writer feels that this statement, reasonable as it appears at first glance, cannot, after all, be so readily accepted. It involves, first, the question of what are comparable structures; and, second, the question of what basis of judging stiffness is to be used. Confining our-

selves to deflections for this latter, as the writer has done, there still remains the question of where the deflections are to be taken. It does not suffice, particularly with arches, to compare only center deflections, for the deflections of non-central points due to non-symmetric loadings are likely to be larger than those at the center, as in the writer's illustration. Such being the case, how shall we combine these? Shall we take the average of deflections at a series of given intervals, or shall we simply compare the greatest deflections wherever occurring? The writer would again refer to his remarks on page 585,* in this connection. Mr. Cilley.

In concluding this reply to Professor Ritter, the writer would suggest that perhaps "the extensive adherence of American engineers to stiff joints in upper chords, as well as rigid attachment of floor-beams, stringers and lateral, portal and sway bracing," is largely due to practical (shop) considerations, together with recognized objections to the use of pins in many cases, especially where adjustable members would be introduced. The abuse of adjustable members has proved to be a serious matter in this country. The writer has been informed of a case where a section of the bottom chord was relieved of its part of the stress and made slack by the over-tightening of counters. And, as is well known, small counters and their connections are often seriously overstrained in this manner.

Perhaps, when a practical system of making connections with thin plates has been developed, a matter which will require much time and experience, even with the best of good will on the part of designers, engineers will sufficiently appreciate its merits in eliminating secondary stresses, to give it preference over the present system of deceptive connections, whose strength is always much less than it is supposed to be, and whose additions to stiffness are only obtained at the expense of notable and unconsidered increases of stress in the main sections.

The comments of Professor Dietz are directed principally to pointing out what numerous factors enter into the final determination of the cost of a structure. To these the writer at once assents, but he discovers in them little or nothing affecting the validity of the arguments he has advanced in the paper. Most of the factors mentioned by Professor Dietz are equally factors with both determinate and indeterminate frameworks, and, therefore, do not influence their comparison, provided only that in any detailed comparison of two designs they are actually made equally factors in the two cases. As to the fact that the superstructure is only an element of the total, as far as it is of moment at all, it is a factor wholly favorable to determinate construction. Nothing like as carefully built and expensive foundations and supports are required for the determinate as for the indeterminate structures. The writer knows of no practical factor except the large (?) expense of

* *Proceedings*, Vol. xxv.

Mr. Cilley. pins which can be cited as adverse to determinate construction, and, aside from the fact previously noted, that their use is not necessary, the saving in cost of erection resulting from it may exceed the cost of manufacture.

Regarding Professor Dietz's detailed comparison of a two and three-hinged arch, the writer begs to note that the placing of the middle hinge below the top chords has very probably deprived the three-hinged arch of a very legitimate advantage it might have had. Moreover, until we know how the sections were determined we cannot know whether the comparison is really fair in this respect. These considerations affect the stated deflection also, but it is in a further respect misleading. It is simply the middle deflection and, as previously noted, very possibly, if not probably, is not the greatest deflection.

Finally, the writer is not inclined to agree at all with Professor Dietz's apparent view that only statistical comparisons of numerous complete projects can be conclusive. There is not now and there probably will not be (in the writer's opinion) within the next century, enough fairly comparable data to enable a definite conclusion to be drawn therefrom respecting the relative merits of determinate and indeterminate frameworks in general. Worked-out cases are almost invariably not fairly comparable. The writer holds that the best, the most reliable, if not the only fair basis of comparison of the relative merits of determinate and indeterminate frameworks must lie in such comparison of ideal structures as he has made. Actual or complete designs are subject to so many irrelevant variations that their comparison necessarily carries with it no general conclusion.

The writer desires to express his appreciation of the brief but very correct summing up of the paper included in the comments of Mr. Joseph Sohn. It is a great help to grasping the main points of such a paper, necessarily somewhat long and complex in its complete form, to have its essential points brought out in so concise and clear a restatement as in the present instance.

The writer is pleased to learn of Professor Jung's adherence to the use of determinate rather than indeterminate systems. Italy has long been a leader in the intelligent application of science to the arts, and notable for her freedom from the old dogmatic beliefs which in many other countries still so largely maintain their hold. The writer well remembers that many of the most lucid and valuable of the modern additions, both to the theory of structures and to the general theory of elasticity, have come from Italy, and believes that in the extension of the use of determinate construction and of intelligent designing Italy will again be in the front.

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PAPERS AND DISCUSSIONS.

EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH. SECOND SERIES.

Discussion.*

By Messrs. E. Kuichling, G. C. Whipple, J. C. Meem, G. S. Williams, George W. Rafter, George H. Fenkell and D. C. Henny.

E. Kuichling, M. Am. Soc. C. E.—One of the most interesting features of this paper is the comparison of the observations made by the authors in 1897 on the large riveted-steel pipe with those made by them in 1899. From this comparison, as exhibited in Table No. 3, they conclude that during the period of two years there has been some decrease in the carrying capacity of this pipe. A similar, although much smaller, reduction is also noticeable in the case of the large wooden pipe, for velocities of 3 ft. and upward, as shown in Table No. 4. The data submitted are very important to hydraulicians, as it rarely happens that the same line of pipe is tested by the same competent and careful observers at different periods of time. **Mr. Kuichling.**

* This discussion (of the paper by Charles D. Marx, M. Am. Soc. C. E., Charles B. Wing, Assoc. M. Am. Soc. C. E., and Leander M. Hoskins, C. E., printed in the *Proceedings* for February, 1900) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)
Communications on this subject received prior to May 25th, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Kuichling. In the speaker's opinion, such a decrease in carrying capacity is always to be expected, and, so far as he has been able to ascertain from the records of reliable gaugings made by others, as well as by himself, it has always occurred, especially in the case of cast and wrought-iron or steel pipes. Two principal causes therefor can be assigned, viz., the gradual formation of nodules of rust, or "tubercles," and the accretion of various organic growths on the inner surface of the conduits. Either of these developments must obviously increase the resistance to the flow of the water in the pipe, and hence also diminish the discharge. The first-named cause is necessarily excluded in the case of a wooden conduit, but in an iron or steel pipe both causes may exist simultaneously.

Rust, or tuberculation, in a water pipe is the result of the action of the water upon the iron, wherever exposed, by defects or abrasions of the protective coating. The area of the imperfection may be very small or even microscopic in size, but its site will sooner or later be manifested by the appearance of a yellowish brown pimple, which gradually increases in size and ultimately assumes relatively large dimensions by spreading laterally over the adjacent sound coating material. Two or more neighboring accumulations of this kind frequently merge into a single large crust, and, when it is forcibly removed, the few minute sources of the supply of hydrated iron oxide are often readily discernible. This oxide evidently attracts or unites with some of the earthy and mineral matter in the water, and the resulting mixture becomes quite coherent, besides adhering more or less strongly to the inner surface of the pipe. In this manner a tubercle is slowly formed, and when the earthy matter preponderates over the iron oxide, it may also become the seat of an extensive organic growth, thereby increasing its size and the degree of obstruction to the flow.

As already stated, the organisms which may lodge and develop in a pipe are of various kinds, and may be of either animal or vegetable character, but adapted to the absence of sunlight and free air. They generally grow on the sides and top of the conduit, and rarely on the bottom, as they appear to dislike the subsidence of fine silt which is apt to occur when the velocity of the water is either greatly reduced, or entirely stopped by entrainment in the branches. Several species of polyzoa, spongilla and algæ, or fungi, were found easily by the speaker in searching for them in dark chambers of reservoir gatehouses and also in the conduits and distributing pipes of several different water-works, but in the latter they were usually restricted to the upper portion of the line or to the vicinity of the source or reservoir. In some cases, however, these growths were found in pipes at a distance of several miles from the head.

An instructive instance of this kind was observed recently in the

new conduit of the Rochester, N. Y., water-works. The northern section thereof is about 9 miles long, and consists of about 1 000 ft. of 36-in. cast-iron pipe, beginning at Rush Reservoir, followed by 38-in. riveted-steel pipe for the remainder of the distance to the city. A number of gaugings of the flow of this section, and measurements of the loss of head in the cast-iron and steel pipes separately, have been made as carefully as was possible, by the same corps of assistants, since its completion in 1894, and the essential results of these experiments are given in Table No. 7. Mr. Kuichling.

TABLE No. 7.—Results of Gaugings of the Northern Section of the New Conduit of the Rochester, N. Y., Water-Works, by the Same Parties, at Different Times between October 17th, 1895, and November 10th, 1899.

No. of gauging.	Date.	Duration of experiment, in hours.	Observed hydraulic grade in the 36-in. cast-iron pipe.	Observed hydraulic grade in the 38-in. riveted-steel pipe.	Observed mean velocity in the 36-in. cast-iron pipe, in feet per second.	Observed mean velocity in the 38-in. riveted-steel pipe, in feet per second.	Deduced coefficient (c) in $v = c\sqrt{rs}$ for the 36-in. cast-iron pipe.	Deduced coefficient (c) in $v = c\sqrt{rs}$ for the 38-in. steel pipe.	Mean age of the conduit in service, in years.
	1895.								
1..	Oct. 17...	5.35	0.0013876	0.0015866	4.204	3.876	129.45	109.35	1.28
2..	Oct. 26...	5.22	0.0015066	0.0016137	4.239	3.908	125.25	109.34	1.30
3..	Nov. 7....	6.38	0.0015034	0.0016180	4.234	3.904	125.25	109.07	1.36
	1897.								
4..	July 28..	10.77	0.0022774	0.0016250	4.128	3.806	99.22	106.11	3.06
5..	Nov. 11..	7.92	0.0048187	0.0015315	4.045	3.730	66.84	107.11	3.35
6..	Nov. 19..	7.08	0.0048535	0.0015334	4.023	3.709	66.24	106.46	3.37
	1898.								
7..	June 4...	8.95	0.0043408	0.0015655	4.034	3.719	70.23	105.65	3.91
8..	Dec. 21...	8.00	0.0037591	0.0015531	4.026	3.712	75.32	105.86	4.47
	1899.								
9..	July 25...	8.00	0.0034455	0.0015349	4.084	3.765	79.79	108.00	5.04
10..	Nov. 10..	8.00	0.0032585	0.0015424	4.077	3.759	81.93	107.58	5.34

It should be noted that the actual mean diameter of the 36-in. cast-iron pipe is 3.0406 ft., as deduced from four measurements of the bore of each separate piece before being set in place, while that of the inside courses of the 38-in. riveted-steel pipe is 3.1667 ft.; also that the corresponding cross-sectional areas are, respectively, 7.2612 and 7.8758 sq. ft., and the lengths between the end piezometers, respectively, 1 094.3 and 45 393.9 ft. The difference in level between the piezometer vessels was determined instrumentally, while the volume discharged by the pipe in the given periods of time was computed from the observed fall in the water surface of the large reservoir at the head of the section, the area for different elevations being known. The water to supply the piezometers was taken in each case from a standard corporation cock screwed squarely into the cast-iron pipe at

Mr. Kuichling. or near its top. Furthermore, at the distributing reservoir in the city, the steel conduit terminates with about 250 ft. of 36-in. cast-iron pipe, but the length thereof which is embraced, up to the lower piezometer, is included in the 1 094.3 ft. mentioned.

When the extremely low value of the coefficient (c) for the 36-in. pipe in Gauging No. 5 became known, it was assumed that an error had been made, and the work was repeated a few days later, with practically the same result, as shown in Gauging No. 6. Special care having then been taken to avoid errors of observation, it was conjectured that the reduction was due to a profuse growth of aquatic organisms in this part of the line, such as was found in the 24-in. effluent pipe of the old conduit at the same locality when a part of it was removed in 1894 for connection with the new pipe. A corroboration of the accuracy of the observations in Gaugings Nos. 5 and 6 was also afforded a few months later by Gauging No. 7.

To ascertain whether this conjecture was correct, an examination of the interior of the pipe was made in September, 1898. The flanged head of a 36-in. special casting in the new gate-house at Rush Reservoir was accordingly removed, and the 36-in. cast-iron pipe was entered and traversed by an assistant for a distance of about 450 ft., while the 38-in. steel pipe was entered at a manhole 3 498 ft. north of the gate-house and traversed for 100 ft. or more in both directions. Owing to the existence of several depressions in the line between these points of entry, and the necessity of soon restoring the new conduit to active service, no further investigations of the intermediate parts of the pipe were made.

On entering the 36-in. cast-iron pipe in the gate-house, extensive organic growths were seen attached to the top and sides, while the lower part was considerably tuberculated. These growths soon increased in magnitude until they formed an almost continuous lining for the entire distance traversed, with frequent large masses hanging down from the top. A number of such pieces were noticed, which were more than 1 ft. square, stretching like curtains across the upper part. The lining, in general, resembled a dense fibrous mat from 1 to 2 ins. thick, and the glossy pitch coating of the pipe was visible only in a few places. Further examination showed that these growths consisted mainly of two species of polyzoa and one of fresh-water sponge, which were identified by Professor Charles W. Dodge, of the University of Rochester, N. Y., as *Plumatella*, *Paludicella* and *Myenia fluviatalis*. At the point where the 38-in. steel pipe was entered, the organic growths and tuberculation had greatly diminished in both size and extent, as only a few scattering patches, from 2 to 8 ins. in diameter, and not exceeding ½ in. thick, were found, along with some small tubercles. The latter were on the lower part of the pipe in the immediate vicinity of the manhole, and doubtless resulted from the abrasion of the coat-

ing by the shoes of the workmen during construction. Elsewhere, the coating was clean, hard and glossy, and apparently as sound as when first applied. Several attempts were made to obtain photographs of the interior of the pipe by flash light at the localities mentioned, but unfortunately all the negatives, on being developed, proved to be dim and imperfect. The foregoing verbal description must therefore suffice to convey an approximate notion of its condition, and hydraulicians will doubtless require no further reasons for the low values of the coefficient (c) in the Chezy formula which were found from the gaugings relating to the comparatively short section of cast-iron pipe. Mr. Kuichling.

From this examination of the conduit, it will be seen that the luxuriant organic growths were limited to perhaps 1 000 ft. or more of the upper part of the line, while the formation of tubercles is possible for its entire length. The reason for such restriction of the extent of the former is obviously found in the fact that the food supply for the organisms, which is contained in the water, is correspondingly limited and becomes practically exhausted beyond a certain distance from the source; hence, in the lower part of the line no organic growths will probably be found. It may therefore be concluded that the gradual diminution of the carrying capacity of a pipe by the development of aquatic organisms on its interior is dependent on the quality of the water, and that when the proper food supply is scanty, little trouble from this cause will be found in a long conduit. On the other hand, there is no such natural limitation to the formation of rust in an iron pipe which is not provided with an absolutely perfect protective coating; and as such a coating has hitherto been practically unattainable, it follows that a reduction of discharge due to this second cause must reasonably be anticipated.

Returning to the case of the Ogden conduit, which was put in service early in 1897, and whose present interior condition is unknown, it is very probable that while no appreciable quantity of organic growths may exist in the steel pipe which forms the lower portion of the long line, there may yet be sufficient tuberculation to account for the reduced discharging capacity which was found by the authors. Two years' time is sufficient for the purpose, when it is remembered that the asphaltic coating was applied by hand, and that numerous imperfections in such work are inevitable, notwithstanding the most rigid inspection. Proof of this conjecture, however, is lacking, and it is therefore to be hoped that the authors will at some future time supplement their present valuable contribution to hydraulics by a third series of gaugings, in which an opportunity to examine the condition of the interior of the conduit will be afforded.

Another interesting point in the paper is the considerably increased loss of head in the Venturi meters, upon whose registration dependence is placed for obtaining the discharge and mean velocity in the

Mr. Kuichling. conduit. The authors have assumed that the loss of head in the meter proper has remained constant, but were unable to verify this assumption. Should it be found by experiment that a change in the frictional resistance of the two tapering sections of riveted pipe which constitute the meter will modify the empirical constant used in computing the discharge, as is by no means improbable, the given numerical values of the Chezy coefficient (c) will also require suitable modification; and hence, in conducting another series of gaugings, the means for testing the registration of the meters should also be included.

Mr. Whipple. G. C. WHIPPLE, Assoc. M. Am. Soc. C. E.—The growth of microscopic organisms in water is, certainly, an important subject, because such growths have a marked effect upon the flow in pipes. These organisms are likely to grow in pipes whenever the water flowing therein contains a sufficient amount of food to nourish them.

The polyzoa, fresh-water sponge, etc., are sedentary forms which grow on the sides of the pipe, and must have their food carried to them. If the water contains the right kind of food to nourish them it is quite likely that they will thrive in any kind of a pipe. The whole subject is chiefly a question of food supply.

For example, in Newton, Mass., which is supplied with a ground-water containing practically no microscopic organisms, an examination of the pipes showed that there were very few polyzoa. In Boston, on the other hand, where the water contains these microscopic forms, many of the pipes were found to be filled with them.

The presence or absence of these organisms can be accounted for, to a great extent, by the microscopical character of the water itself. It is possible that the water flowing in the Ogden pipe-line may be so free of the smaller microscopic organisms that none of the sedentary organisms are present in any part of the pipe-line, while the water may be of such a character as to cause tuberculation of the steel pipe, without having any such chemical action on the wooden pipe.

It would be interesting to know whether the authors made any examination of the quality of the water flowing through the pipe, and whether any opportunity was offered them to examine the pipe-line with reference to the presence of polyzoa, fresh-water sponge and other similar organisms.

Mr. Meem. JAMES COWAN MEEM, Assoc. M. Am. Soc. C. E. (by letter).—In comparing the steel pipe tests for 1897 and 1899, it is noted that with clean pipes, in 1897, the value of the coefficient of roughness n was not only much lower, but was also much more uniform than in 1899, when the pipe had undoubtedly become tuberculated with rust and growths. It is undoubtedly a valuable increment to our knowledge to find that with tuberculated pipes the value of n seems to decrease in proportion to the rise of velocity, below, and up to 3 ft. per second; while, for practical

purposes, it may be considered as constant under the same conditions for clean pipes under all velocities. Mr. Meem.

This latter point is borne out by the 1899 experiments on the wood-stave pipes, which show a constant value for n for all velocities; and it is further borne out by some crude experiments made by the Sewer Department of Brooklyn, in 1896, on the 15-ft. brick sewer on Fourth Avenue, which also showed a practically constant value for n under varying velocities.

The comparison between the 1897 and 1899 experiments on the wood-stave pipe shows a marked decrease in the value of n after two years' use. This doubtless shows that organisms require more, as Mr. Whipple suggests, than that their food be brought to them, but that they must also have a "lodging place," which is probably furnished in the steel and iron pipes by the commencement of rust, and points to the further fact that wood pipe seems to wear smoother with usage. Arthur L. Adams,* M. Am. Soc. C. E., states that the growths in wood pipe are not found to exist where the pipe is constantly full and under pressure. It is probable, then, that the presence of decay only in this pipe furnishes the nucleus about which these organisms grow.

Comparing these experiments with those noted or made by Mr. Adams,† it is seen that the value of n is much lower in the stave pipes of smaller sizes. Thus, in the 72-in. pipe in the paper it is 0.0131, while in the experiments by Mr. Adams on 14-in. and 18-in. pipes, $n = 0.010$ for the 18-in., and 0.0107 to 0.011 in the 14-in. pipe. This confirms substantially what the writer has long believed to be true, that in pipes of small diameters, all else being equal, the value of n is lower than for pipes of large diameter. Comparison of the experiments referred to, on the 15-ft. brick sewer in Brooklyn (which though crudely made showed the value of n to be between 0.013 and 0.015), with some other experiments‡ made by T. C. Hatton, M. Am. Soc. C. E., of Wilmington, Del., on small pipes (in which the value of n was shown to be materially lower than 0.013) partially confirms this conclusion, although not decisively or definitely enough to be of value, as they were not compared under the same conditions.

The summary of these deductions is as follows:

(*a*) In smooth pipes of the same size and kind the value of n is practically constant under all velocities.

(*b*) In tuberculated pipes under all high velocities the value of n is constant, for the same sizes and conditions; while

(*c*) In tuberculated pipes, for the same conditions, under low velocities, the value of n increases inversely with the velocity.

(*d*) In clean pipes the value of n probably decreases with the diameter of the pipe, other conditions being the same.

* *Transactions*, Am. Soc. C. E., Vol. xli, p. 84.

† *Transactions*, Am. Soc. C. E., Vol. xli, p. 55.

‡ Published, the writer believes, in *The Engineering Record*.

Mr. Meem. In assigning the value of n, in estimating or calculating the velocities in proposed pipes and conduits, it is, of course, beyond the range of possibility to test the proposed pipes under the exact conditions of usage; or to find experiments fitting each case exactly. The writer, therefore, proposes a factor of safety, as in construction, which may be used at the discretion of the engineer. As our tables and diagrams are usually calculated for a value of 0.015 for n, we may take this as a basis on which the following factors of safety are suggested for adoption:

(1) For steel-riveted or cast-iron pipes, calculated to run under low velocities, or which are inaccessible for cleaning, add 20% to the total discharge in making the final calculations for sizes.

(2) For steel-riveted or cast-iron pipes, under high velocities, or which are accessible for cleaning, add 10 per cent.

(3) For brick, concrete and pipe conduits, under all velocities, and of over 3 ft. in diameter, or wood-stave pipe alternately wet and dry, add nothing.

(4) For small pipes of all kinds except iron or steel, and wood-stave pipes of over 3 ft. diameter, and running full at all times, deduct 10 per cent.

(5) For wood-stave pipes of less than 3 ft. diameter, constantly full and under pressure, deduct 20 per cent.

Mr. Williams. GARDNER S. WILLIAMS, M. Am. Soc. C. E. (by letter).—The authors are to be congratulated upon having performed the most accurate series of experiments yet published upon wooden-stave pipe. Their observations upon the riveted pipe do not, however, appear to be quite so reliable, either from unsuspected sources of error in the methods and means of observation, or, much more probably, on account of conditions in the pipe line itself which prevent the general application of the results obtained from it; so that, while the conclusions which may be drawn from them may not be suitable for common use, the experiments themselves, for the particular case in hand, may be in every way as accurate and deserving of confidence as those upon the upper section of stave pipe, which the writer believes will compare favorably with the best pipe experiments yet published, and which appear to be of wide application. The reasons for this differentiation of the results will be discussed later. Before proceeding with this question, however, there are other points in the paper which should be considered.

The authors (page 111) argue that in order to have the gauge readings vitiated by accumulations of air in the connections, there must be present a quantity sufficient to fill the cross-section of the pipe completely. It appears to the writer that any mixture of air and water must have a less specific gravity than water alone, and hence, if even minute bubbles of air be present in a connection rising to a

gauge, the effect will be to cause the gauge to read high. As this very principle has long been utilized in the old forms of air-lift pumps, it does not seem necessary to discuss it further than to say that if the connecting pipe slopes downward to the gauge, the result will be a low reading, and that while very few, if any, of the present observations give evidence of such a source of error, the fact remains that observations may be affected by air which does not fill the cross-section of the pipe. Mr. Williams.

On the same page the authors refer to observations with piezometers connected at various points in the circumference, from which they appear to conclude the correctness of the very interesting theoretical discussion of this subject presented in their former paper.*

On this point the writer has to submit that he knows of experiments with very delicate apparatus for measuring the pressures at different points of the circumference of a pipe in which water was flowing, which showed that the pressures not only are different at different points, but that they change sign relatively to each other as the velocity changes, one becoming alternately greater and less than the other at the opposite extremity of the same diameter. As it is expected that these experiments will shortly be presented to the Society, with some others on the flow of water in pipes, the writer does not wish to go into detail here, but it seems to him entirely safe to conclude that the hypotheses assumed by the authors in their former discussion were contrary to fact, and that in the present case, had they had more delicate apparatus, they would probably have found a difference in the readings at the several points of the circumference.

In reference to the general results, it is always desirable to have some criterion by which to judge of the accuracy of experiments as they are published, aside from that of the personality and reputation of the observers; and the writer has for some time been in the habit of applying to his own observations the following:

Assuming that the loss of head varies as the square of the velocity, the former must also vary as the first power of the velocity head; whence, in any series of experiments covering more than one velocity, if the velocity heads and the corresponding losses of head be used as co-ordinates, theoretically, the points so located should fall in a straight line which may be expected to pass through the origin; and the deviation of the points so plotted from such a line will indicate the relative merit of the several observations. It will at once be appreciated that while some form of curve can usually be made to pass through and fit fairly well a great many observations, it is not such an easy matter to fit a straight line to them.

This criterion has been applied by George H. Fenkell, Jun. Am. Soc. C. E., to nearly all the so-called standard pipe experiments hitherto

* *Transactions*, Am. Soc. C. E., Vol. xl, p. 522.

Mr. Williams. published, and it is somewhat surprising to see how many supposedly reliable observations are nothing more than crude approximations. It is hoped that Mr. Fenkell's work may soon be given to the public in order that the records may be purged of a considerable number of experiments which should never have been published, and that the relative merits of others may be estimated properly.

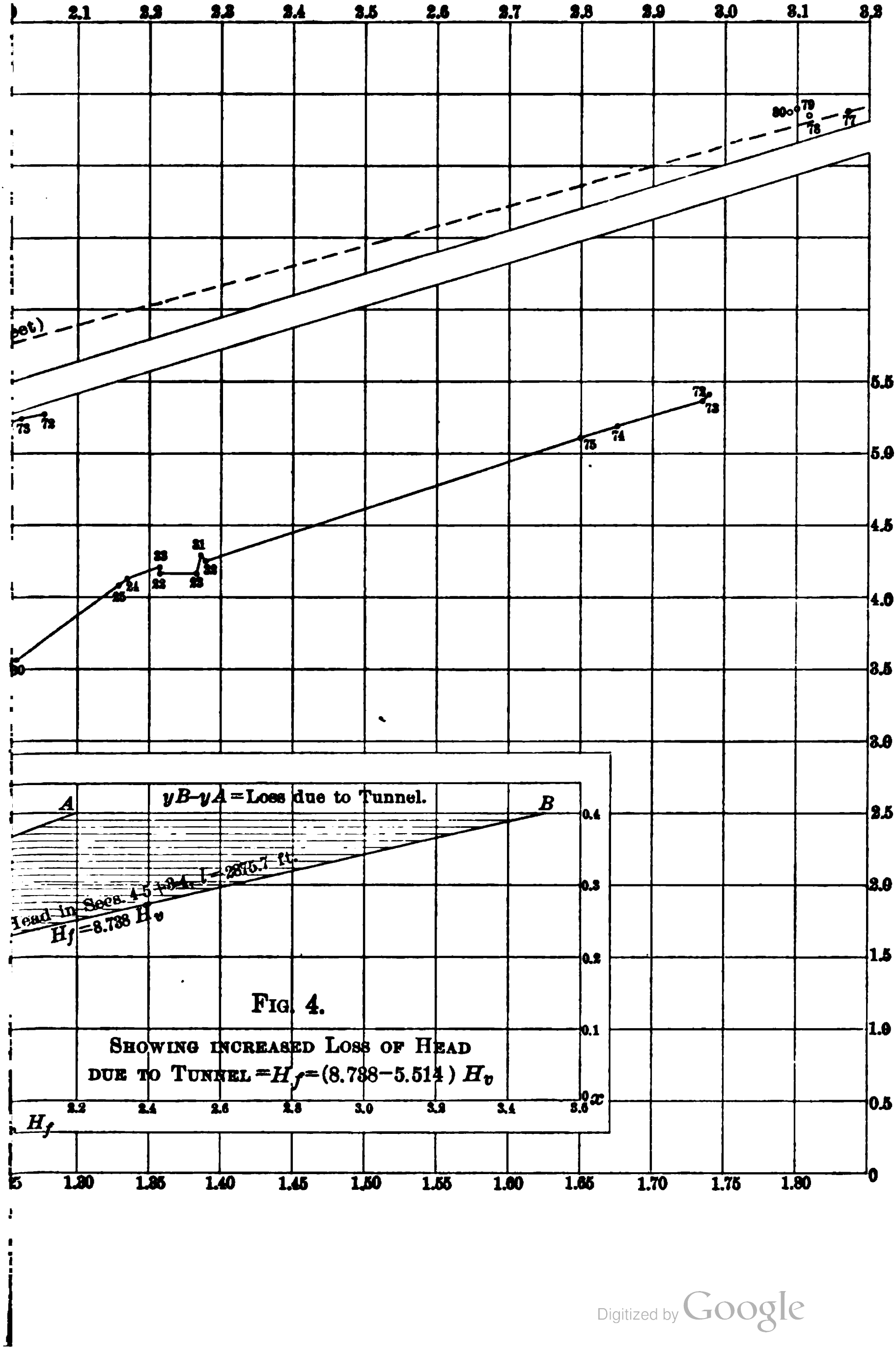
An application of this criterion to the observations of the authors in 1897 showed that the high degree of accuracy claimed by them, based upon reductions by the method of least squares, was frequently imaginary.

In Fig. 1, Plate XXIII, the writer has plotted the observations presented in Table No. 1, of the present paper, and also the data of the previous series of experiments. To avoid unnecessarily complicating an already rather intricate diagram, the straight lines to which the observations should conform have not been drawn, but if a fine thread be stretched along the line of observations for the long section of stave pipe, the excellence of the results will be at once apparent, and if the equation of the mean line be computed by summations of the ordinates and abscissas we obtain $H_f = 1.9794\ H_v$, where H_f is the loss of head and H_v is the velocity head, measured in feet.

Similarly, if the thread be stretched along the points for the short section of stave pipe, which are united by the fine broken line it will not only be seen that the inclination of the line is not the same as in the other case, but also that the points do not fit as well, and that, in fact, this series of observations appears to be very little better than those of 1897, which are indicated by letters and joined by the dotted lines.

Observations Nos. 40, 41, 56, 57, 58, 59, 37, 38, 48 and 44, seem to show too low a loss of head as compared with the velocity head, while 35, 36, 50 and 51, show the opposite condition. Excepting the four last named, it appears that in this section of pipe the observations at low velocity indicate a much lower resistance than corresponding observations on the longer section, a result which, while it does not accord with the former observations on this section, as indicated by A, B, C, D, E, G, H and K, is seemingly correct.

Remembering this condition of affairs in Section 3–4, we now consider the Tunnel Section 4–5, the observations upon which are plotted in Fig. 2, Plate XXIII. By stretching the thread here it will be seen that the observations fail to have the semblance of conformity to the straight line, and it appears that all the observations at low velocities show abnormally high losses of head, exactly the opposite condition to that observed in the section next down stream, already considered. These facts seem to indicate that the piezometer at No. 4 gave a low reading, particularly at low velocities. Without attempting to account for this at present we will combine the total losses of head in Section 4–5 with the simultaneous ones in Section 3–4, and plot the combined


yB–yA = Loss due to Tunnel.
A
B
H_f = 8.738 H_v
L = 2875.7 ft.
FIG. 4.
SHOWING INCREASED LOSS OF HEAD DUE TO TUNNEL = H_f = (8.738 − 5.514) H_v

result to the velocity heads in the stave pipe, which plotting is shown in Fig. 3, Plate XXIII. In this figure, the mean line obtained by summation of ordinates has been drawn, the group of observations at the lowest velocity being given half the weight of the others, and Nos. 42, 50, 36 and 51, have been rejected. It is to be remarked that Nos. 36, 50 and 51, seem to contain errors of observation or reduction, but as they were grouped by the authors with others this point does not attract especial attention in their plottings, although quite evident here. The close approximation of the remaining observations, except perhaps No. 46, is quite remarkable, although there is still a tendency for the low velocities to show excessive loss of head, which has no counterpart in the observation on the long section of stave pipe. Mr. Williams.

The following explanation of the phenomenon might not have been so obvious, but for the fact that a similar one was encountered recently by Messrs. E. C. Murphy and C. C. Torrance, their observations being corroborated by the writer, in the Hydraulic Laboratory of the College of Civil Engineering of Cornell University, where such precautions were taken as to leave no possibility of questioning the results.

All are familiar with the fact that when water flows from the bottom of a vessel to a pipe of smaller cross-section, a spiral motion is generated, as may be seen at any time in the ordinary washbowl. It is, therefore, aside from other proofs, quite safe to assume, when there is an abrupt change of section in a pipe line from a larger to a considerably smaller one, that a similar spiral motion will be set up, and it needs no extended argument to prove that the higher the velocity, the longer the pitch of the spiral, which latter will keep lengthening as the water flows on in a straight pipe, the particles tending continually to move more and more nearly in straight lines. It is well known, further, that velocity head is convertible into pressure head and *vice versa*. It becomes apparent, therefore, that if the pressure head be measured a short distance below a point where a reduction of cross-section of the flowing stream has taken place, we may expect to find that the true velocity of a particle of water, the flow at that point being in a spiral, will exceed the rectilinear velocity in the direction of the axis, and will also exceed the true velocity further along the pipe where the pitch of the spiral has been increased, whence the pressure head at the up-stream point will be reduced by the excess of velocity head there, and it is entirely possible to conceive, what actually occurs in the Venturi meter, that the following length of pipe might show a gain of pressure rather than the loss to be expected from frictional resistances. This is, apparently, exactly what takes place in the Ogden pipe on which Piezometer No. 4 is 23 ft. down stream from the tunnel mouth where a contraction takes place from a section 9 ft. square to a 6-ft. circular one. The lower the velocity, the shorter the pitch of the spiral. Therefore, at low velocities, the ratio of the true or spiral velocity of

Mr. Williams. the particles to the rectilinear velocity will be much greater than at high velocities; hence the pressure head will be much lower relatively to that at other points, as observed by the authors. The gradual straightening of the spiral results in readings farther down much more nearly in accord with those above and unaffected by the contraction. It may be remarked that this spiral motion, while apparently reducing the loss of head in the section below the contraction, must actually increase it; and a very interesting point is raised as to the rapidity with which the filaments straighten themselves out, or, in other words, how far below a contraction we must go to get a normal pressure reading. The fact that the lower observations, as plotted in Fig. 3, Plate XXIII, show an excessive loss of head, indicates that a distance of 2 733 ft. in the 6-ft. pipe is not entirely sufficient to hide its effects from observation with the mercurial measuring apparatus used here. How much farther the effect would be noticeable, with a water column or some more delicate apparatus, is an open question.

To determine the effect of the tunnel, it is evident that it is not proper to proceed as the authors have done and from the difference shown by Piezometers Nos. 4 and 5 take the computed loss in 67 ft. of stave pipe. The nearest to a correct proceeding, possible with the data at hand, is to take from the loss of head shown between Piezometers Nos. 5 and 3 the loss due to an equal length of stave pipe as deduced from the observations on the long section, when the effect of the tunnel will be seen to follow the same law as that of other resistances to flow, *i. e.*, to increase as the square of the velocity, and not in the manner indicated by the authors upon pages 122 and 123.

On Fig. 4, Plate XXIII, the result is shown graphically, the ordinates between the lines *O A* and *O B* showing the apparent increased loss of head due to the tunnel. From this it appears that the loss is nearly 60% greater with the tunnel as it is than it would be with the stave pipe continued through it.

It may be proper to call attention to the fact that in Figs. 1, 2 and 3, Plate XXIII, the velocity heads are plotted in inches of water, not in feet, as the writer happened to have at hand a transformation curve for velocities to velocity heads in this unit. In Fig. 4, Plate XXIII, the velocity heads are plotted in feet.

Considering the steel pipe, it appears that the two groups of observations, Nos. 22, 23, 24, 25, 31, 32 and 33, and Nos. 72, 73, 74 and 75, fall very well upon the straight line through the origin, while the others show a higher loss of head than the line fitting the observations named. If we fit a line to the observations from Nos. 18 to 11, as plotted, the observations at the lowest velocities still show losses of head which are too high, so we have either to assume that the loss of head in riveted pipe does not vary as the square of the velocity, or else look for a cause for a low reading in Piezometer No. 1 or a high

one in No. 2. The apparently close and accurate instrumental work done by the authors at the other gauges does not give much weight to the probability of errors of observation here, so that the cause is rather to be sought in the line itself. The data published seem hardly sufficient for the case, although the curvature in the line may possibly set up enough spiral flow to cause the lower gauge to read low. The authors themselves are best qualified to discuss this question. As to the possibility that the law in riveted pipe may be different from that in cast-iron or wood, it may be said that the observations of Emil Kuichling, M. Am. Soc. C. E., on the Rochester conduits and of J. Waldo Smith, M. Am. Soc. C. E., on the East Jersey Water Company's lines, do not indicate anything of the sort, nor do observations upon highly tuberculated pipes, nor, as pointed out previously, those on the tunnel section. The writer, therefore, has come to the conclusion regarding the riveted pipe work, already expressed, that although the observations may be equally accurate, the conditions are not such as to make the results as valuable as are those on the stave pipe. Mr. Williams.

From the apparent fact that the observations on the short section of stave pipe are influenced by the tunnel effects which are subject to quite wide variation, according as the flow is increased or decreased to produce the velocity under consideration, it seems rather questionable to attempt to determine the effect of age upon the capacity of the conduit by comparisons between the observations in 1897 and those of 1899 on this section. A similar criticism may be made as to the steel pipe, though to a somewhat more restricted degree.

The effect of spiral currents upon the registry of the meters is another interesting consideration. From the velocity head plottings, both in the stave pipe experiments and those on the riveted pipe, which were not simultaneous, it appears that there was a similar irregularity of results with velocity heads between 0.30 and 0.45 in. This, of course, points to a common cause, which seems most likely to be found in the meters. The position of the meters being such that the water enters by a Y and a curve,* it is easy to conceive that, at some velocities the spiral motion generated at the Y may be reduced by the curve, and at others intensified there, and this might easily account for the irregularity referred to, as a greater or less spiral motion would reduce or increase the difference of head between the inlet and the throat, which seems to indicate that, after all, the Venturi meter, like many another type, is only absolutely reliable when used under exactly the conditions at which it was rated.

In this connection it may be well to add that when observations are plotted, as the writer has done in these cases, it is possible and proper to reject inaccurate ones, and to deduce coefficients from the

* *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 279.

Mr. Williams. points on the line rather than from the actual observations, thus eliminating, to a large extent, the individual inaccuracies. Having proceeded thus far, it only remains to drop the ancient Chezy and the modern Kutter formulas, and adopt one with an approximately logical foundation, to get the subject of hydraulics into a shape consonant with nineteenth century progress.

Mr. Rafter. GEORGE W. RAFTER, M. Am. Soc. C. E. (by letter).—This paper is an elegant sequel to the previous one, and presents, in compact form, data of considerable value to hydraulicians. The minute detail with which the authors have described their methods and appliances will be, without doubt, very satisfactory to those who find themselves unable to judge of the value of a paper when presented on the broad lines of useful results, without reference to the methods used in reaching them. The authors are to be congratulated on having given apparently everything necessary for intelligent judgment.

The foregoing reflections are not in any way suggested by deficiencies in the paper, but refer to its minute fulness. As regards hydraulic methods and appliances, every possible question seems to have been answered, thus leaving one free to discuss the broad problem of increase of friction head with advancing age in large riveted steel, wood-stave or other water conduits, and without special reference to the detail of the present experiments, which, taken in conjunction with the authors' previous paper, may be assumed to illustrate the hydraulic conditions of the Ogden pipe in considerable detail.

Broadly, we may assume that many water conduits decrease somewhat in carrying capacity with advancing age. The paper indicates that for the Ogden pipe, this decrease is greater in steel pipe than in wood. Indeed, the wood pipe is shown to have increased in carrying capacity with advancing age. The reasons for these differences are not given, although it seems clear enough that an investigation of flow through a water conduit which reveals such marked changes in carrying capacity, as well as differences between two kinds of pipes, as are here indicated, ought to be accompanied by a study of the causes for such changes and differences.

Engineers sometimes err in not tracing out final causes, especially in cases like the present one where fields of knowledge, outside of engineering pure and simple, require to be traversed in order to reach an adequate solution. Nevertheless, the writer does not criticise the paper because of such omission, but merely mentions it as a suggestion for future work.

In 1891, when the writer presented his paper on "The Hydraulics of the Hemlock Lake Conduit of the Rochester, N. Y., Water-Works,"* drawing the conclusion, from certain discharge measurements made in

* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 13.

1890, that the high values of c in the expression $v = c\sqrt{r\,s}$, previously Mr. Rafter. used in computing flow through large mains, were not justified by the facts, his conclusions as to the relatively low values of c really applying to riveted-steel conduits, were somewhat questioned. Time, however, is a great clarifier, and not only are the writer's views as to the lower values of c now universally admitted, but, since 1891, considerable energy has been expended in showing why, in large steel conduits of some age, it is impossible that any view other than the writer's could possibly apply. On this line, a theory of gradual deterioration has been built up, and many facts have been gathered, tending to substantiate it.

While the writer has no desire to controvert the view that many water conduits have decreased in carrying capacity with age, he nevertheless wishes to point out that this result is by no means universal—the Ogden wood-stave pipe is a case in point—and that there are undoubtedly a considerable number of water conduits a long time in use, which are to-day discharging at full capacity substantially as much water as when first placed in service.

Speaking broadly, the evidence, as it now stands, apparently indicates that steel-plate conduits are particularly subject to decrease in carrying capacity with increased age. So far as the writer can determine, there seems to be a practical difficulty in coating the built-up-plate conduits, which, in some degree, militates against their usefulness. It is true that coated cast-iron pipes have also shown decrease in carrying capacity, but not, the writer concludes, to such an extent as the wrought-iron or steel-plate conduits.

As regards deficiencies in the coating, the writer is unable to add very much to his discussion of Mr. FitzGerald's paper on "Flow of Water in 48-In. Pipes," presented to the Society in 1896, and it is proposed, therefore, to give here a brief account of what may be termed the biological causes for the decrease in carrying capacity, referred to.

So far as present information goes, difficulties of this sort have been referred to two classes of animal life, namely, fresh-water sponges and fresh-water polyzoa. The fresh-water sponges have been introduced to the Society by Mr. FitzGerald in his two papers, "Spongilla in Main Pipes"* and "Flow of Water in 48-In. Pipes," † but without giving any account of them. So far as the writer now recollects, aside from a brief reference in the discussion of his paper on "The Hydraulics of the Hemlock Lake Conduit of the Rochester, N. Y., Water-Works," the polyzoa have not been considered before this Society as the cause of decrease in carrying capacity of water conduits. Hence, the writer feels justified in giving a brief statement of the habits of the sponges and polyzoa.

* *Transactions*, Am. Soc. C. E., Vol. xv, p. 887.

† *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 241.

Mr. Rafter. Knowledge of the American forms of fresh-water sponges was extended greatly by the monograph * of Edward Potts, who has described and figured a considerable number of species, and his paper may be referred to as embodying practically all that we know about the American forms of these interesting animals.

Some of the fresh-water sponges may form incrustations like those described by Mr. FitzGerald in his paper entitled, "Flow of Water in 48-In. Pipes," although, thus far, only one or two species have been certainly identified as offending in these particulars, as, for instance, *Spongilla lacustris* at Boston, and *Meyenia fluviatalis* at Rochester. It is possible that on further study other forms of fresh-water sponge may be found to be giving rise to this trouble, although, aside from one or two forms, such supposition is rendered slightly improbable because close conduits do not seem to be the natural home of many species.

According to Mr. Potts, fresh-water sponges do not differ in constitution and general appearance from marine sponges, except that the fresh-water forms are characterized by the presence of certain seed-like bodies called gemmules or statoblasts, and which are in effect eggs from which new colonies issue. These gemmules are not found in marine sponges.

The gemmules are nearly spherical and about $\frac{1}{50}$ in. in diameter. They are sometimes found floating freely in waters inhabited by fresh-water sponges. As to the methods of germination, Mr. Potts' monograph may be referred to for the detail.

Most of the fresh-water sponges are characterized by a more or less vivid green color, which, however, is not universal, but, according to Mr. Potts, is closely dependent on the quantity or quality of the light received. A sponge which has germinated away from the light will be nearly white, gray or cream colored. If brought into full sunlight, it gradually becomes green, finally attaining a bright vegetable green. Some species, as for instance, *Meyenia leidyi*, are stated to never become green. On this account, *Meyenia leidyi* may be expected to grow in water conduits, where there is entire absence of light. This species is further characterized by a persistent habit through which the growths of successive seasons rise one above another, forming series of thin laminæ. In this way it sometimes builds up smooth rounded prominences of compact texture. The fact that it is sometimes found at considerable depths, may also be taken to indicate that this species may be found on the interiors of water conduits.

The fact that most of the sponges are naturally of a bright green color must, so far as present information goes, be taken to indicate

* "Contribution Towards the Synopsis of the American Forms of Fresh-Water Sponges, with Descriptions of those Named by other Authors in all Parts of the World," by Edward Potts. *Proceedings*, Academy of Natural Sciences of Philadelphia for April-August, 1887.

that their natural habitat is in places sometimes subject to exposure to light, although many of the species undoubtedly prefer subdued light. Professor Lankester has shown the occasional occurrence of chlorophylloid green coloring matter in the tissue of animals, among others in fresh-water sponges, in *Meyenia fluviatalis*. A fact of this character tends strongly to show that the natural habitat is to be found in localities at any rate receiving some light, although it ought not to be overlooked that several species grow in places where the light is subdued. Without going into the subject extensively, the writer will for the present merely state his opinion that this is, with many species, largely a question of convenient attachment to a fixed support, rather than a necessary elementary condition. At any rate, the presence of the chlorophylloid coloring substance, in either plants or animals, implies exposure to light.* Mr. Rafter.

The fresh-water sponges are widely distributed throughout the United States, Mr. Potts stating that he has examined *Spongilla fragilis* from at least 32 localities in 18 States; *Spongilla lacustris* from 26 localities in 16 States; *Meyenia fluviatalis* from 25 localities in 14 States; and *Tubella pennsylvanica* from 18 localities in 11 States. That the fresh-water sponges are subject to considerable modifications under varying environments is indicated by the statement of Mr. Potts that hardly any two specimens were found exactly alike in their so-called typical features.

The place of the fresh-water sponges, in the animal kingdom, is found in the sub-kingdom, Protozoa. They are, therefore, allied to the Infusoria and the Rhizopods, the lowest forms of invertebrate animals.

The fresh-water polyzoa are a much higher form, being found in the sub-kingdom, Mollusca, and hence related to the Cephalopods, Gastropods, and other relatively advanced forms of the Molluscan sub-kingdom. They possess a complex alimentary system, with an œsophagus, stomach and intestinal tract. They also have a nervous system and well-developed muscles. Some species grow attached, while others are free-floating. Fixed forms are usually found immovably fastened by their ectocysts to the lower surface of submerged stones or floating timbers. In their fully developed state, the polyzoa live in colonies, a large number of individuals, each with its own digestive tube, tentacles, nerve ganglion, muscles, generative organs, etc., each reproducing itself either by gemmation or fixed statoblasts, while all continue organically united in a single group. Reproduction is either by budding or by statoblasts. When from buds, they grow from the side of an adult polypide. Statoblasts may be either fixed or free. The free statoblasts are the founders of new colonies, while buds merely increase the number of individuals in each colony.

* For reference to Professor Lankester's determinations of chlorophyl in minute animal forms, see Sacks' "Text Book of Botany," p. 767.

Mr. Rafter. Generally, the polyzoa do not thrive in places subject to the direct action of sunlight. Their natural habitat seems to be in dark places. Undoubtedly, some of the species may exist in entire absence of light, although whether they thrive vigorously in such places is not definitely known. *Cristatella* is, however, an exception. According to Professor Alman, this polyzoon delights in exposure to the full influence of the sun, and may be seen basking upon the upper side of submerged stones, or creeping over the stems of aquatic plants in lakes and ponds.* Mr. Hyatt, however, points out that *Pectinatella* apparently prefers strong light in July and August, but cannot stand it at all in October. In the summer months it is found abundantly distributed in exposed shallow waters, but in the fall it disappears from such locations, and can then be discovered only in shaded places several feet below the surface. Colonies of *Pectinatella* frequently grow several feet in diameter.

Many species of the fresh-water polyzoa show considerable color, although not always a chlorophyllaceous green. *Paludicella* is, however, an olive green color. Its single filaments are from $\frac{1}{70}$ to $\frac{1}{80}$ in. in diameter. It is somewhat unlikely that with an olive green color predominant in most normal specimens, the natural habitat is found in absolute darkness. *Fredricella* may be cited as a species which apparently prefers nearly absolute darkness. According to Mr. Hyatt, this form is only found in the darkest places. *Plumatella* is another form apparently preferring light, and which is frequently found growing from the ends of water grasses, without any protection whatever from light and heat.

Aside from the influence of light on the development of color, there are two other facts which tend to show that the fresh-water polyzoa are not likely to grow naturally in water conduits. In searching for them along streams and about ponds, they are only occasionally found in rapidly moving water, their preference being apparently for very slowly moving or quiet water. Nor, have they usually been found in water of more than 3 or 4 ft. in depth, the under side of a floating plank or timber being usually the best place to find them. It is somewhat doubtful, therefore, if they readily stand heavy water pressure, and, hence, so far as the present information goes, we should only accept well-attested statements as to their presence in pressure conduits.

Another difficulty is as to the method of attachment. With a perfectly smooth interior surface, there is apparently no way by which polyzoa can attach themselves. But if we assume a broken coating with ragged projections, the case becomes very simple. Floating filaments or germinating statoblasts would be easily caught, and large

* For convenient English literature of the fresh-water polyzoa, refer to "Alman's Fresh-Water Polyzoa," published by the Ray Society, in 1856, and to Hyatt's Observations on the Polyzoa, in *Proceedings* of the Essex Institute, Salem, 1866-68.

colonies ultimately developed. As regards fresh-water sponges, the Mr. Rafter. conditions of attachment are somewhat similar, although the sponge has a power of secreting a gluey substance with which it attaches itself to stones in running water, and which might enable it to become attached in slowly moving water within a conduit. In the case of the 48-in. pipe examined by Mr. FitzGerald, it appears that there was a break in the coating at nearly every point where a sponge was attached. But whether this was due to the action of the sponge or was the original cause of attachment, is unknown. Reasoning from purely *a priori* considerations, however, it appears somewhat probable that breaks in the coating were the prime cause.

The foregoing very brief account of the fresh-water sponges and polyzoa, in connection with decrease of carrying capacity of water conduits, is sufficient to show that there are several difficulties to be surmounted before a safe theory of deterioration, which shall have universal application, can be successfully formulated. Nevertheless, the writer does not wish to be understood as saying that in some cases decrease in carrying capacity may not proceed from these causes, although, broadly, the evidence is yet too indefinite to form a safe general hypothesis. That is to say, the writer accepts the well-observed special cases, but is not satisfied that the growth of such forms is the universal explanation. The known preference of the polyzoa for slowly moving or quiet water, together with the development of color masses akin to chlorophyl in the fresh-water sponges and possibly, also, in some forms of polyzoa, militates somewhat against the view that they develop other than accidentally under the conditions of rapid movement and absolute darkness prevailing in water conduits.

As regards chlorophyl in these animal forms, it is not believed that it performs quite the same functions that it does in green plants. It is intended to go no further than to state that these animal forms do develop a chlorophylloid substance which is undoubtedly dependent upon the quantity of light received. For such forms a water conduit does not appear to be the natural habitat.

Nevertheless, the fact of finding polyzoa attached to the interiors of water conduits, the same as fresh-water sponges, ought not to be ignored. Very interesting questions are opened up by such a discovery, and some engineer-biologist, endowed with the scientific spirit and a fair stock of patience, has here an opportunity to elucidate a problem which is not only scientifically interesting, but which has very important commercial bearings. But those who have not pursued working biology to some considerable extent may well await, in this matter, the opinion of the qualified expert, because probably in the entire range of science there is no one place where either preconceived opinions, or those founded on insufficient data, are so likely to be modified by study and experience as in considering the different manifestations of

Mr. Rafter. life and the modifications to which it is subject, under varying environment. Realizing this truth, the writer, therefore, stands ready, on the presentation of acceptable evidence, to modify whatever in the nature of opinion is here expressed.

Mr. Fenkell. GEORGE H. FENKELL, Jun. Am. Soc. C. E. (by letter).—There is little doubt that the authors have obtained for large wood and riveted pipe the most satisfactory set of experiments ever published. The generalized results obtained, however, for 1897 and 1899, show a considerable variation, and it is impossible to ascribe the cause of these discrepancies to the effect of leaks in the apparatus or to anything else connected with the manipulation of the instruments or the readings of the same. Some of the single observations are obviously in error, as will be seen later, but, on the whole, they show very careful work, and most of the discrepancies must be looked for, either in the effect of curvature on the flow, the tunnel, the Venturi meters, or in the reductions of the observations.

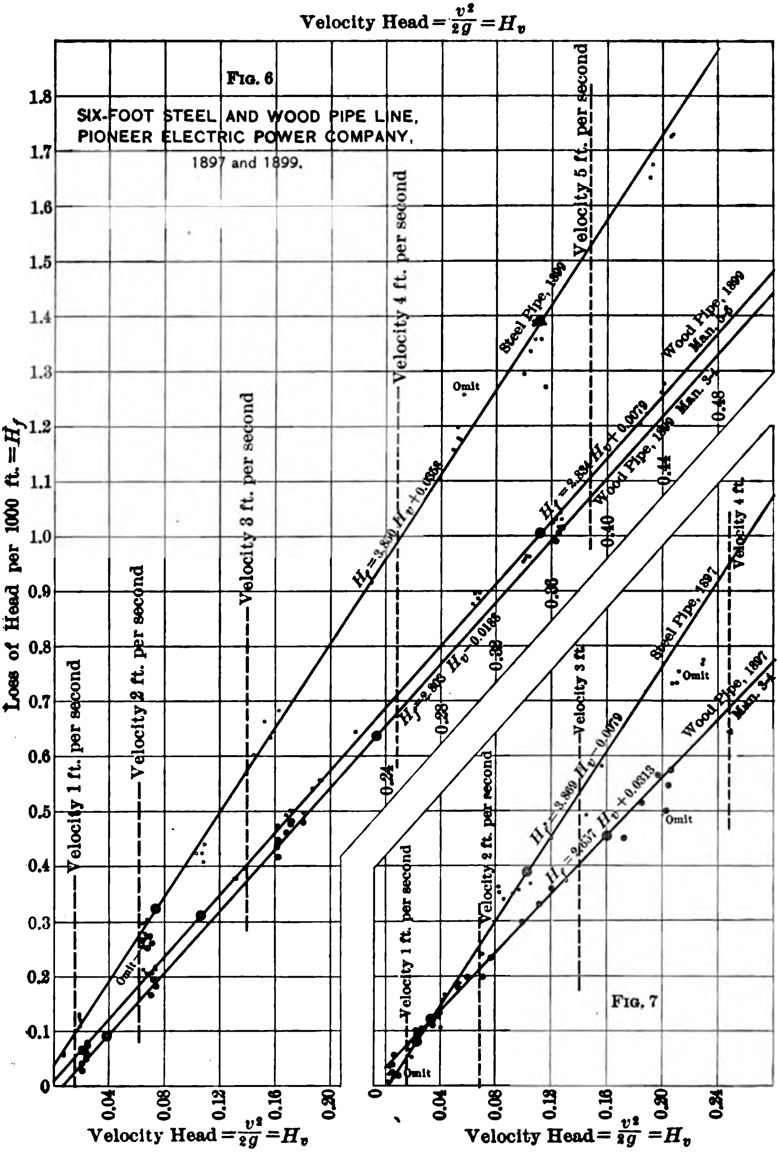
The writer will not attempt to discuss the first three reasons given, but will confine himself entirely to a criticism of the reductions, the results of which the authors have published in Tables Nos. 3 and 4.

A method of judging the relative value of experiments on the flow of water through pipes, and enabling the student to omit such observations as prove to be unreliable, having given the velocity and the loss of head, first proposed by Gardner S. Williams, M. Am. Soc. C. E., and since used by him and Clarence W. Hubbell, Jun. Am. Soc. C. E., on a wide class of work, is used by the writer.

If $v^2 = 2\,g\,h$, $h = \frac{v^2}{2\,g}$ = velocity head = H_v. If, for each given velocity, the velocity head is calculated and is plotted with the loss of head (H_f), or friction, a straight line through the origin should be the resultant, if the loss of head varies as the square of the velocity.

In Figs. 6 and 7, all observations in each series are shown in this manner. It is a well-known fact that the most satisfactory way to study the relations existing between a series of two quantities is by plotting on squared paper; and it is generally customary to average the results thus obtained (if a straight line) by means of a fine thread, which can be moved about until the best average seems to have been found. The writer knows of no discussion on this subject, and it seems to have hardly been considered by hydraulic engineers. In this discussion the average line is found by first finding the center of gravity of an entire series, by dividing the sum of the ordinates and the sum of the abscissas by the number of observations. The series is then divided into two parts by the center of gravity. The center of gravity of each of these two parts is found and is plotted on Figs. 6, 7, 8 and 9 with three concentric circles. This method was first suggested to the writer by Mr. C. W. Hubbell. These two points, which

Mr. Fenkell.


Velocity Head $= \frac{v^2}{2g} = H_v$
FIG. 6
SIX-FOOT STEEL AND WOOD PIPE LINE,
PIONEER ELECTRIC POWER COMPANY,
1897 and 1899.
Loss of Head per 1000 ft. $= H_f$
Velocity 1 ft. per second
Velocity 2 ft. per second
Velocity 3 ft. per second
Velocity 4 ft. per second
Velocity 5 ft. per second
Steel Pipe, 1899
Wood Pipe, 1899 Man. 5-6
Wood Pipe, 1899 Man. 3-4
$H_f = 3.830\ H_v + 0.0358$
$H_f = 2.834\ H_v + 0.0079$
$H_f = 2.808\ H_v - 0.0188$
Omit
Steel Pipe, 1897
Wood Pipe, 1897 Man. 3-4
$H_f = 3.869\ H_v - 0.0079$
$H_f = 2.657\ H_v + 0.0313$
Velocity 3 ft.
Velocity 4 ft.
FIG. 7
Velocity Head $= \frac{v^2}{2g} = H_v$
Velocity Head $= \frac{v^2}{2g} = H_v$

Mr. Fenkell. are in line with the center of gravity of the entire series, determine the position of the average line, and only in rare instances, with ordinary experiments, would this line pass precisely through the origin.

It is probable, however, that all errors due to observation, personal error of the observer, calibration of instruments and leaks or air bubbles in the gauge connections, are nearly constant for all velocity heads, and hence for all velocities, and if the average line for a series of points is moved parallel to itself until it passes through the origin, it will then be the average line if all errors are eliminated from the observations.

It is also evident, after plotting all points, that a few are in error to such an extent that they had better be dropped entirely. Most of these are mentioned by the authors as being of a doubtful nature, and in Figs. 6 and 7 they are all marked "omit." In the experiments on the wood pipe, for 1897, Manometer 3–4, Observations 35 and 46 are so treated. In the experiments on the wood pipe, for 1899, Manometer 3–4, Observations 35, 36, 50 and 51 were dropped, while with Manometer 5–6 all are considered reliable. Observations 16, 15, 10, 29 and 28 are omitted from the experiments on the steel pipe, for 1897, and Observation 30 from the 1899 experiments. The lines deduced from the reliable points are drawn in Figs. 6 and 7, and their equations are as follows, H_v referring to velocity head and H_f to loss of head per thousand feet, or friction:

SIX-FOOT WOOD PIPE.

Manometer 3–4, 1897 .... $H_f = 2.657\ H_v + 0.0813$

" " 1899 .... $H_f = 2.808\ H_v - 0.0183$

" 5–6, 1899 .... $H_f = 2.884\ H_v + 0.0079$

SIX-FOOT STEEL PIPE.

Manometer 1–2, 1897 .... $H_f = 3.869\ H_v - 0.0079$

" " 1899 .... $H_f = 3.850\ H_v + 0.0858$

These equations are the average lines as plotted, and the constant denotes the distance on the vertical axis that the line passes from the origin. If each one is moved parallel with itself until it passes through the origin, they then drop the constant and become:

SIX-FOOT WOOD PIPE.

Manometer, 3–4, 1897 .... $H_f = 2.657\ H_v$

" " 1899 .... $H_f = 2.808\ H_v$

" 5–6, 1899 .... $H_f = 2.884\ H_v$

SIX-FOOT STEEL PIPE.

Manometer, 1–2, 1897 .... $H_f = 3.869\ H_v$

" " 1899 .... $H_f = 3.850\ H_v$

These equations show the relation existing between the velocity head and the loss of head, and, consequently, with a given velocity, by multiplying its velocity head by the coefficient of H_v, as given in

Mr. Fenkell.

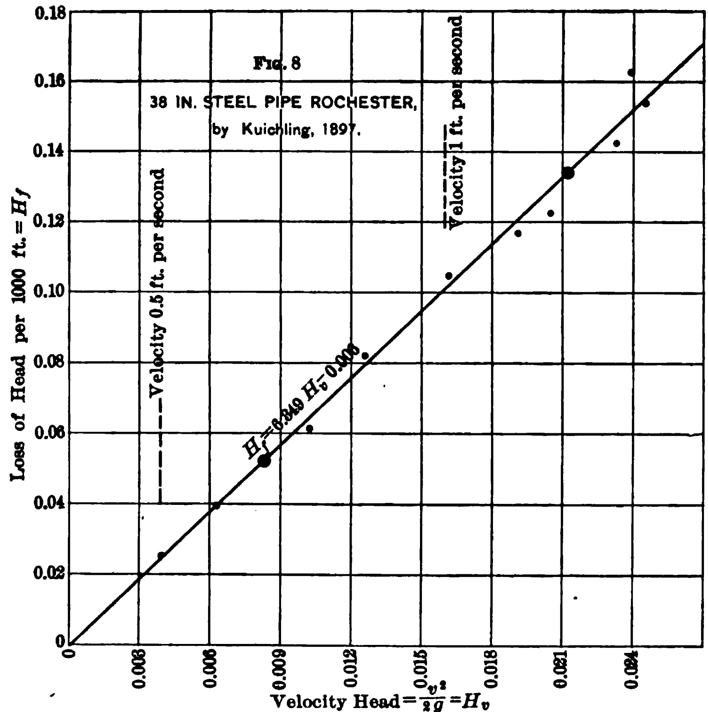


FIG. 8
38 IN. STEEL PIPE ROCHESTER,
by Kuichling, 1897.
Velocity 0.5 ft. per second
Velocity 1 ft. per second
$H_f = 6.349\ H_v - 0.006$
Loss of Head per 1000 ft. $= H_f$
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0
0.003
0.006
0.009
0.012
0.015
0.018
0.021
0.024
Velocity Head $= \frac{v^2}{2g} = H_v$

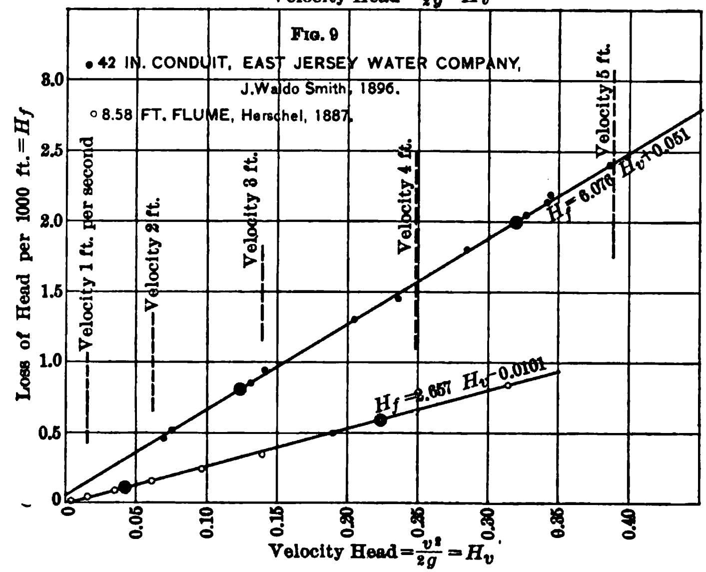


FIG. 9
• 42 IN. CONDUIT, EAST JERSEY WATER COMPANY,
J. Waldo Smith, 1896.
○ 8.58 FT. FLUME, Herschel, 1887.
Velocity 1 ft. per second
Velocity 2 ft.
Velocity 3 ft.
Velocity 4 ft.
Velocity 5 ft.
$H_f = 6.076\ H_v + 0.051$
$H_f = 2.657\ H_v - 0.0101$
Loss of Head per 1000 ft. $= H_f$
3.0
2.5
2.0
1.5
1.0
0.5
0
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
Velocity Head $= \frac{v^2}{2g} = H_v$

Mr. Fenkell. any of the foregoing equations, the result will be the loss of head per thousand feet for that particular pipe. Table No. 8 has been deduced from these equations, and shows the results as compared with those given in Tables Nos. 3 and 4, and the values of c, in the formula $v = c\sqrt{r\,s}$, as compared with those given by the authors.

TABLE No. 8.

Six-Foot Wood Pipe. Generalized Results.

	1897.					1899.					
	Manometer 3-4.					Manometer 3-4.		Manometer 5-6.			
Velocity.	Velocity Head. H_v.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. Table No. 4. H'.	c. From Column 3.	c. From Table No. 4.	Loss per 1 000 ft. H_f.	c. From Column 7.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. Table No. 4. H'.	c. From Column 9.	c. From Table No. 4.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1.0	0.0155	0.041	0.066	127.3	100	0.043	124.5	0.044	0.049	123.2	116.0
1.5	0.0349	0.093	0.123	127.3	110	0.098	124.5	0.099	0.106	123.2	118.7
2.0	0.0621	0.165	0.200	127.3	115	0.174	124.5	0.176	0.184	123.2	119.9
2.5	0.0971	0.258	0.292	127.3	119	0.272	124.5	0.273	0.284	123.2	120.8
3.0	0.1398	0.371	0.400	127.3	122	0.392	124.5	0.396	0.404	123.2	121.4
3.5	0.1902	0.505	0.527	127.3	124	0.533	124.5	0.539	0.548	123.2	121.7
4.0	0.2485	0.660	0.678	127.3	125	0.697	124.5	0.704	0.712	123.2	122.0
4.5	0.3144	......	......	......	......	0.881	124.5	0.891	0.898	123.2	122.2
5.0	0.3882	......	......	......	......	1.088	124.5	1.100	1.105	123.2	122.4
5.5	0.4697	......	......	......	......	1.317	124.5	1.331	1.335	123.2	122:5

Six-Foot Steel Pipe. Generalized Results.

	1897.					1899.			
	Manometer 1-2.					Manometer 1-2.			
Velocity.	Velocity Head. H_v.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. H'.	c. From Column 15.	c. From Table No. 3.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. From Table No. 3.	c. From Column 19.	c. From Table No. 3.
(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
1.0	0.0155	0.060	0.055	105.5	110	0.060	0.100	105.6	81.6
1.5	0.0349	0.135	0.121	105.5	111	0.134	0.177	105.6	92.0
2.0	0.0621	0.240	0.220	105.5	110	0.239	0.277	105.6	98.0
2.5	0.0971	0.376	0.356	105.5	108	0.374	0.405	105.6	101.3
3.0	0.1398	0.541	0.510	105.5	108	0.538	0.570	105.6	102.4
3.5	0.1902	0.736	0.673	105.5	110	0.732	0.765	105.6	103.2
4.0	0.2485	0.961	0.863	105.5	111	0.957	0.987	105.6	103.8
4.5	0.3144	......	......	......	......	1.210	1.237	105.6	104.3
5.0	0.3882	......	......	......	......	1.494	1.516	105.6	104.7
5.5	0.4697	......	......	......	......	1.808	1.824	105.6	105.0

It is evident that if the straight line passes through the origin, $H_v \propto H_f$. If $v = c\sqrt{r\,s}$, $c = \dfrac{v}{\sqrt{r\,s}}$. As $v^2 \propto H_f \propto H_v$, and as r is constant for each size, c will remain constant for all velocities in the same pipe, as shown in the table. This method of reducing the observations shows considerable decrease in the carrying capacity of the wood pipe, while that of the steel pipe is slightly increased. This increase is so small, however, that it should not be considered, as it represents a fineness hardly justified by the experiments. It does show, however, that the carrying capacity of the steel pipe, as nearly as can be calculated, was the same in 1899 as in 1897. Mr. Fenkell.

TABLE No. 9.—Rochester 38-Inch Steel Pipe, 1897.

Emil Kuichling, M. Am. Soc. C. E.

Velocity.	Velocity Head. H_v.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. Annual Report.	c. From Column 3.	c. From Annual Report.
(1)	(2)	(3)	(4)	(5)	(6)
1.33897	0.02788	0.1518	0.1629	113.2	109.07
1.25425	0.02448	0.1550	0.1545	113.2	113.41
1.22236	0.02323	0.1474	0.1418	113.2	115.41
1.15065	0.02057	0.1305	0.1229	113.2	116.63
1.10751	0.01907	0.1211	0.1172	113.2	114.98
1.02060	0.01618	0.1027	0.1048	113.2	112.08
0.89790	0.01255	0.0797	0.0819	113.2	111.53
0.81413	0.01030	0.0653	0.0615	113.2	116.49
0.63742	0.00630	0.0400	0.0397	113.2	113.66
0.50525	0.00396	0.0251	0.0254	113.2	112.58

TABLE No. 10.—42-In. Conduit, East Jersey Water Company, 1896.

J. Waldo Smith, M. Am. Soc. C. E.

No. of Observation.	Velocity.	Weight.	Velocity Head. H_v.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. "115 Exp."	c. From Column 5.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
259..............	2.21	*B.*	0.0758	0.461	0.53	110.1
260..............	3.02	*B.*	0.1416	0.86	0.93	110.1
261..............	3.90	*B.*	0.2362	1.44	1.45	110.1
262..............	4.69	*C.*	0.3408	2.08	2.14	110.1
263..............	4.59	*B*	0.3271	1.99	2.04	110.1
264..............	4.70	*A.*	0.3415	2.08	2.18	110.1
265..............	4.29	*A.*	0.2858	1.74	1.80	110.1
266..............	3.63	*A.*	0.2045	1.24	1.31	110.1
267..............	2.91	*A.*	0.1315	0.80	0.85	110.1
268..............	2.10	*A.*	0.0686	0.42	0.47	110.1
270..............	4.99	*A.*	0.3868	2.35	2.40	110.1

Mr. Fenkell.

TABLE No. 11.—8.58-Ft. Flume at Holyoke, 1887.

Clemens Herschel, M. Am. Soc. C. E.

No. of Observation.	Velocity.	Velocity Head. H_v.	Loss per 1 000 ft. H_f.	Loss per 1 000 ft. "115 Exp."	c. From Column 4.	c. From "115 Exp."
(1)	(2)	(3)	(4)	(5)	(6)	(7)
501	0.5	0.0039	0.0104	0.0078	106.1	126.5
502	1.0	0.0155	0.0412	0.0320	106.1	116.6
503	1.5	0.0349	0.0927	0.0822	106.1	112.7
504	2.0	0.0621	0.1650	0.1532	106.1	110.3
505	2.5	0.0971	0.2580	0.2421	106.1	108.3
506	3.0	0.1398	0.3714	0.3520	106.1	107.7
507	3.5	0.1902	0.5054	0.4902	106.1	106.9
508	4.0	0.2485	0.6603	0.6520	106.1	106.2
509	4.5	0.3144	0.8354	0.8350	106.1	105.6

Fig. 8 represents the observations on the 38-in. steel conduit, at Rochester, made by Emil Kuichling, M. Am. Soc. C. E., in 1897,* and the writer believes them to be the best experiments on large riveted pipe ever made at low velocities. In Table No. 9 the loss per thousand feet and values of c are compared with those published by Mr. Kuichling. This table was deduced in the same manner as those for the 6-ft. wood and steel pipes.

Fig. 9 shows a few of the results on the East Jersey Water Company's 42-in. riveted pipe,† obtained by J. Waldo Smith, M. Am. Soc. C. E. Although a large number of observations were made on this company's conduits, they were taken on so many different lengths, with but few series on the same pipe through a range of velocities, that only a part of them is capable of being worked up in this manner. Many of them show wide discrepancies; that which is shown being the best one published. Fig. 9 also shows, in like manner, the generalized results, taken from an average line, of the experiments on an 8.58-ft. flume,‡ 152.88 ft. long, by Clemens Herschel, M. Am. Soc. C. E. As the single observations were not published, it is impossible to tell the amount of averaging necessary to produce the published results. A comparison of published results and those calculated by the writer on these pipes is shown in Table No. 11.

In order to test a set of experiments by this method, it is necessary to have a considerable range of velocities on the same length of pipe, and the first series by the authors was the first set so taken on large stave pipe. Figs. 6 and 7 show the best sets of experiments on large riveted pipe, to date, which can be tested by this method. It is to be

* "Annual Report, Executive Board," Rochester, 1897.

† "115 Experiments on the Carrying Capacity of Large Riveted Metal Conduits, up to 6 ft. per second of Velocity of Flow," by Clemens Herschel, M. Am. Soc. C. E.

‡ *Transactions*, Am. Soc. C. E., Vol. xvii, 1887, and "115 Experiments," by Clemens Herschel.

regretted that the experiments on the Astoria wooden and riveted pipe, by Arthur L. Adams,* M. Am. Soc. C. E., are not capable of such comparison. Mr. Fenkell.

D. C. Henny, M. Am. Soc. C. E. (by letter).—In his discussion of the first series of experiments the writer touched upon some elementary factors which in the first paper had, in his judgment, failed to receive the attention they deserved. Being cognizant of the fact that the first experiments were to be supplemented and extended, he hoped that an opportunity would be sought to remove more fully the doubt which seemed justified regarding some of these points. In this respect, the writer confesses to some disappointment, the more severe as he realizes the scarcity of available information and the perhaps unduly enhanced value which is likely to be accorded to individual experiments. Mr. Henny.

Diameter.—In the present paper the authors have continued to assume the interior diameter of the stave pipe at 72½ ins. No mention is made of any check, by actual measurement, upon this assumption, which, for reasons stated, the writer considered as probably incorrect. He can now add that, last January, he made a hasty examination of the wooden pipe at the point where it crosses the trestle, at Station 276, at which point Manometer No. 3 was located. He found that while the assumed interior diameter, 72½ ins., corresponds with the projection of thread at each lug of 3¾ ins., the actual projection exceeded this in every case. On a few of the bolts he measured a projection of 8 ins., made possible only by the placing of dozens of washers or fillers under the nuts. Whether the bolts themselves were longer than stated by Mr. Goldmark in his paper† on the Pioneer Power Plant the writer had no means at hand for determining. The exposed portion of the pipe had the appearance of having been re-cinched, after completion, to stop leakage, it being observed incidentally as an interesting fact, that many of the pressed-steel lugs had become badly deformed, the side walls having generally bent inward until in some cases they touched at the top.

The possibility of such reduction in diameter emphasizes the uncertainty, well known to wooden-pipe builders, of basing an estimate of the probable diameter of a stave pipe upon the original width of the individual staves, as appears to have been done by the authors.

It is not contended that the apparent reduction of diameter here observed renders a similar excessive reduction likely where the pipe is buried. The probabilities are the other way, yet it leaves it pertinent to ask why the authors have omitted to present measurements of the outer circumference of the pipe, readily obtainable at the various points where the pipe passes over bridges and trestles. Even if such

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 1.

† *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 270.

Mr. Henny. measurements could not be accepted as true averages they would have defined more clearly the relative importance of the possible error due to this cause. The writer believes that at the point examined by him the pipe may well have been less than 71 ins. in diameter, and have had an area close to 5% less than that assumed.

It was contended by the authors in their first paper * that there was no need of such measurements: First, because similar uncertainty as to diameter would exist in other stave pipe, which the writer cannot admit, because experience makes it possible, in the construction of stave pipe, to approximate closely to the desired diameters; and second, because the possible error attributed to this cause would be insignificant for the purpose of explaining the disparity in results of the Ogden and previous experiments, which is also objected to as imposing an illogical and irrelevant limitation upon the accuracy of the experiments.

Presence of Air.—The writer has failed to find any information as to the air which had accumulated at summits where it could be blown off. Check valves to be depressed with a bar afforded the only means for releasing air, and these did not occur at all summits, judging from the profile † presented by Mr. Goldmark. In this respect the long section of stave pipe is not as thoroughly protected as the shorter section with the tunnel relief shaft at its upper end. Air at summits, where it cannot be blown off, does not, in the writer's opinion, necessarily reveal its presence through any irregularity in the results of experiments.

Presence of Sediment.—It was considered highly improbable by Mr. Goldmark that sediment could have affected seriously the first experiments on the short section of stave pipe, because the pipe had been in use only a few months previous to the time the experiments were made. This argument does not hold good in the present instance, two years having elapsed. The importance of this matter has become forcibly impressed upon the writer's mind by a recent occurrence which is of interest in this connection.

The light wooden trestle carrying the 52-in. inverted stave-pipe siphon, on the line of the Santa Ana Canal‡ across Deep Cañon, near Redlands, Cal., was originally designed for a load equal to the weight of the pipe and water immediately over it. No account was taken, so far as the writer is aware, of the additional load due to thrust from vertical curvature, amounting, as was shown in his discussion of the paper mentioned, to over 40% of the load figured on. The trestle settled considerably at the time of filling the pipe, and in the course of years showed further dangerous signs of weakness, until, a short time

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* *Transactions*, Am. Soc. C. E., Vol. xl, p. 557.

† *Transactions*, Am. Soc. C. E., Vol. xxxviii, p. 255.

‡ *Transactions*, Am. Soc. C. E., Vol. xxxiii, Plate xvii.

ago, a portion of it collapsed, carrying a part of the pipe down with it. It then became apparent that, for about 60 ft. in length, the pipe had gradually filled with fine sand, which had become indurated, and which had left only a small passage for the water in the lower reaches of the siphon. The sediment had added to the load, and had probably contributed materially to the disastrous result. So far as this incident concerns the present discussion and the possibility of the formation of sediment in pipe lines generally, it should be understood that during the last three or four years, only a very small flow of water had passed through the pipe, which had thus been converted into a long settling basin; and that the sand-box at the upper end of the canal may not have been effective or properly operated. Yet, considering all circumstances, the probability of a serious amount of sediment forming in this siphon did not seem great. Mr. Henny.

The Ogden pipe takes water directly from a similar mountain stream; its intake is believed to be unprovided with any sand-trap or settling basin, and the writer understands that the small percentage of the total available power ordinarily utilized calls for only a low velocity in the pipe.

Unless definite information can be presented, tending to prove the pipe to have had a clear section, it is necessary to fall back upon a comparative study of results, and with this end in view Table No. 12 has been deduced.

TABLE No. 12.—Experiments on Six-Foot Stave Pipe.

Group.	Velocity, in feet per second.	Value of c in the Chezy Formula.		
		Short Section.		Long Section.
		1897.	1899.	1899.
		Deduced from average curve.	Calculated from average of observations for each group	
(1)	(2)	(3)	(4)	(5)
J............	1.175	104	155	117
N............	1.244	106	137	118
L............	2.126	116	116	121
I............	2.144	116	118	119
K............	3.239	123	127	121
M............	3.324	123	124	121
T............	4.845	....	125	123

Table No. 12 gives the values of c for the velocities indicated in Column 2. Column 3 gives these values as deduced from the average curve deduced from the 1897 experiments on the short section of

Mr. Henny. stave pipe. Column 4 gives the values found on the same section of pipe in 1899, and Column 5 gives the values found on the long section of pipe in 1899.

The grave disparity which may be observed in the results for low velocities is sufficient to cause their rejection. For velocities above above 1.5 ft. per second the results agree in a satisfactory manner.

As regards the presence of sediment, the comparison gives undoubted weight to the assumption that if no sediment were present in the short section of pipe in 1897, there probably was none in 1899. From the close agreement of the experiments on the short and long section (Columns 4 and 5), the further deduction may be made that the long section must also have been practically free from sediment. The writer believes, however, that this chain of reasoning is not sufficiently strong to depend on for conclusions, where the physical facts in the case, so far as understood by the writer, rather favor the inference that some unknown amount of sediment may have been present in the pipe. Blow-off gates, as argued before, would, upon their first discharge, have afforded the means of establishing the presence or absence of sediment at least at or near the points where attached. If they have been utilized for this purpose the writer has failed to find any mention of it by the authors.

Curvature.—Experiments to determine the effect of sweeping curves upon the loss of head in pipe lines are yet to be undertaken. Measuring the loss of head due to short and sharp curves is not believed to furnish a safe guide in this respect. Short elbows alternating with long tangents in a pipe line are likely to have only a local effect upon the motion of the particles. Skin friction in the straight reaches will tend to oppose the disturbance and will favor, and probably reproduce, approximately parallel and rectilinear motion. With long curves, the centrifugal force, while smaller in amount, has longer time to act and to establish a *régime* of its own for each particular curve, which, whatever it may be, whether a generally rotating motion, or arbitrary eddying, is sure to increase the frictional loss. With comparatively short tangents intervening, the new *régime*, after being established, will extend into the next curve, and be either confirmed or destroyed to make place for another general scheme of movement, and every change of motion involves additional loss.

With a pipe line, such as was available to the authors, having hardly any true tangents, it was impracticable to study the effect of curvature independently. Moreover, it must be admitted that most wooden pipe lines have a considerable percentage of easy curvature. Being familiar with a large number of stave pipe lines now in use, the writer will say, however, that he knows of none where the percentage of true tangents, especially if expressed in diameters, is so small as would appear to be the case with the Ogden pipe line, judging from the

plan and profile shown in Mr. Goldmark's paper. And especially is this true if the Ogden pipe line be compared in this respect with stave pipe lines elsewhere, experiments whereon have been recorded. Speculation on this subject may be fruitless, yet it emphasizes the necessity of entering this field analytically before hydraulicians can hope to make material progress. Mr. Henny.

General Remarks.—The writer wishes to reiterate, that possible presence of air and sediment, as well as probable deficiency in diameter, if causing error at all, have produced cumulative results in increasing the value of *c*; while an unusually high percentage of length of curves tends in the same direction. Whether the combined effect may be deemed sufficient to bring these experiments in line with those previously undertaken with smooth surfaced conduits of various classes, or whether a serious doubt should be entertained as to the accuracy of the Kutter formula as applied to smooth-bore pipe, can hardly be settled satisfactorily until additional light be thrown upon this complex subject. In the absence of additional and convincing information to the contrary, the writer inclines to the former proposition. Moreover, the new experiments on the stave pipe, taken by themselves, constitute a forcible confirmation of the accuracy of the Kutter formula so far as it accounts for change in velocity.

The experiments on the steel pipe strengthen the belief that the Kutter formula offers no advantage over the simple Chezy formula.

On the important subject of effect of time on the carrying capacity, the experiments shed some valuable light. Table No. 4 is, in this respect, not as conclusive as Table No. 12, since the latter permits of a comparison of the experiments of 1897 and 1899 on the same section of stave pipe. The value of *c* shows no diminution during the two years' interval. With the steel pipe, the diminution in carrying capacity is very marked, considering the short time the pipe has been in use, and while the earlier experiments indicate a superiority in the carrying capacity of stave pipe over steel pipe, of about 12% for 3½ ft. velocity, this difference appears to have increased, in the course of two years, to 20 per cent.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

THE ALBANY WATER FILTRATION PLANT.

Discussion.*

By Messrs. G. L. CHRISTIAN, JOHN C. TRAUTWINE, JR., and ALLEN HAZEN.

Mr. Christian. G. L. CHRISTIAN, Assoc. M. Am. Soc. C. E. (by letter).—This paper is a valuable addition to the literature of water filtration. The large death rate from typhoid fever before the construction of the filter, and the material reduction in the deaths from that cause alone would justify the expenditure necessary for its installation. But, when all the inhabitants use the filtered water it would seem that the death rate should be decreased even more than it has been.

The writer was interested in the cracks in the concrete and brick masonry caused by the low temperature. He was also interested in Mr. Rafter's remarks on that subject, having had a similar experience some years ago while engaged on a reservoir in which there was a straight masonry wall approximately 475 ft. long. The height of this wall from the bottom of the reservoir to the under side of the coping was 26 ft. Its thickness was 4 ft. at the top and 13 ft. 6 ins. at the bottom.

It was built of second-class rubble masonry, the constituents being a freshly quarried gneiss of good quality and a mortar composed of 2 parts sand to 1 of a standard American Portland cement.

The rock was all cleaned and washed immediately before being laid, and the work was done by good masons, of experience in that line of work, and who were under constant supervision. The wall was com-

* Continued from February, 1900, *Proceedings*. See November, 1899, *Proceedings* for Paper, by Allen Hazen, M. Am. Soc. C. E., on this subject.

pleted during the latter part of August, and, as cold weather approached, it was examined carefully every few days. Mr. Christian.

Early in December the thermometer fell suddenly to 12° Fahr., and on that day cracks were noticed, as shown in Fig. 18. They extended

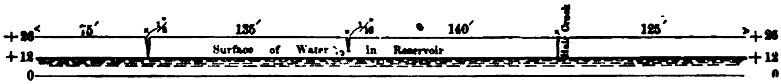


Fig. 18.

through the wall, were about at right angles to it, and generally followed the joints. They never widened, but, on the contrary, closed gradually as the water rose in the reservoir and the warm weather approached, finally closing so tightly that only by the closest scrutiny could they be seen, even though each place had been marked carefully in order that it might be found readily.

John C. Trautwine, Jr., Assoc. Am. Soc. C. E. (by letter).—To the writer this paper has a special interest as a source of information and a basis of comparison, in connection with the several large plants now being designed for the filtration of the Philadelphia supply. Mr. Trautwine.

After innumerable reports upon filtration, by Councilmanic Committees, by local organizations,* by the writer, as Chief of the Bureau of Water, and by his predecessor, Mr. John L. Ogden, and after years of fruitless discussion and inaction, the present Mayor, Hon. Samuel H. Ashbridge, who took his seat on April 3d, 1899, secured from Councils, on April 20th, an ordinance providing for "the employment of three experts relative to the improvement, filtration and extension of the water supply," who were "to act in conjunction with the Director of the Department of Public Works, Chief of the Bureau of Water and Chief of the Bureau of Surveys in examining and reporting upon the question."

On May 8th, the Mayor announced the appointment of Messrs. Rudolph Hering, of New York; Samuel M. Gray, of Providence, and Joseph M. Wilson, of Philadelphia, who, on September 15th, presented a report recommending, in conclusion:

1. The adoption of that project by which the waters of the Schuylkill and Delaware Rivers, taken within the city limits, are purified by filtration.

2. The immediate improvement of the existing plant, in accordance with the detailed recommendations of their report.

Among the improvements recommended was the restriction of waste by the introduction of meters.

In their *résumé* and conclusions the experts say:

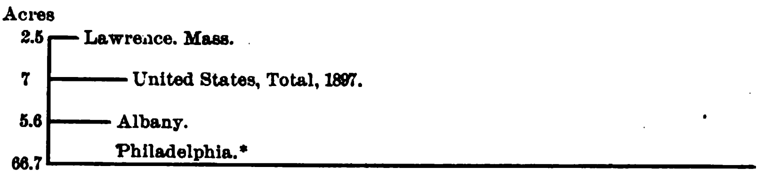
"We consider that, at present, a daily supply of 200 000 000 galls., being 150 galls. per capita, is a very liberal allowance. We recommend that this quantity of pure water be immediately provided for."

* Including the Woman's Health Protective Association, who submitted a report by Mr. Jos. B. Rider and, later, one by Mr. Allen Hazen.

Mr. Trautwine. This involves a material restriction of the consumption, which is now variously estimated at from 220 000 000 to over 275 000 000 galls. daily.

The striking features of the Albany plant are, perhaps, on the one hand, its immensity when compared with previously existing American slow filtration plants, and on the other hand, its insignificance (if the word may be used without offence), in comparison with the magnificent installation recommended by the experts for Philadelphia.

Prior to the construction of the Albany plant, with its aggregate filter-bed area of 5.6 acres, that at Lawrence, Mass., with a corresponding area of 2.5 acres was much the largest slow plant in America, and in 1897 the aggregate area of all those plants of which the writer could learn did not reach 7 acres (Fig. 19).



*For a supply reduced to 200 million gallons per day, as recommended by the experts, at 3 million gallons per acre per day.

COMPARISON OF AREAS OF FILTER BEDS.

FIG. 19.

Upon visiting the Albany plant during its construction, in May, 1899, the writer was much impressed with the great extent of the operations in progress, indicated by the photographic views accompanying the paper, and it was therefore a startling reflection that these vast provisions would be insufficient to supply one of the four 20 000 000-gall. pumps at the Queen Lane (Philadelphia) pumping station, which station represents barely one-fifth of the aggregate nominal capacity of all the Philadelphia pumps, and that the daily pumpage at Philadelphia* was nearly fifteen times greater than the capacity of the Albany plant.

Even with the restricted consumption contemplated by the experts, the Philadelphia filtration plant will be nearly, if not quite, the largest in the world, and certainly very much larger than any other under a single control.

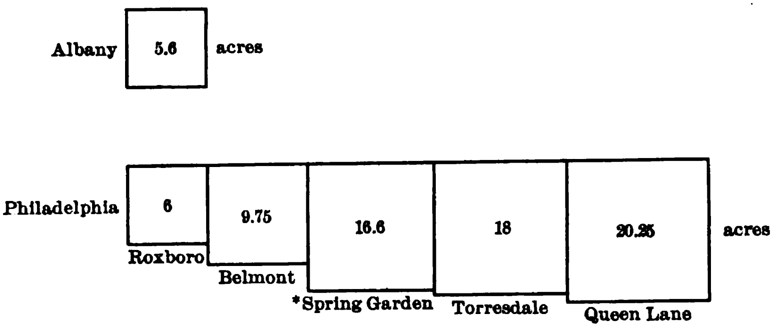
Philadelphia is supplied from six stations, the smallest of which (Roxborough) has a nominal daily capacity of 24 500 000 galls., and is credited, in the Bureau report for 1898, with an average daily pumpage of over 20 000 000 galls., as against 15 000 000 galls. nominal daily

* Two hundred and twenty million U. S. galls., as estimated by the experts. By plunger displacement, after allowance for slip, the pumpage, as stated in Report of Water Bureau for 1898, was 275 000 000 galls.

capacity, and 12 500 000 galls. actual consumption, for the Albany plant. Mr. Trautwine.

The aggregate bed area of the four slow plants recommended by the experts for immediate construction for Philadelphia is 54 acres, and, in addition to this, a rapid or "mechanical" plant of 50 000 000 galls. daily capacity was recommended for the Spring Garden System.

The filter-bed areas of these four plants, and the equivalent slow-bed area of the rapid plant recommended for the Spring Garden System* are shown graphically, and compared with that of the Albany plant, in Fig. 20.



*"Rapid" Plant, 50 million gallons daily, taken as equivalent to $\frac{50}{3}$=16.6 acres of slow beds.

COMPARISON OF AREAS OF FILTER BEDS. ALBANY AND PHILADELPHIA.

FIG. 20.

The construction of the Albany plant happens most opportunely for the authorities in charge of the Philadelphia work, who may profit by it, not only as a pattern, but, if time permits, as a most useful monitor, showing where modifications may be advisable in the Philadelphia plants in order that they may be more perfectly adapted to the conditions there existing. Indeed, unless the differences in conditions between the two places are studied most carefully, the Albany plant will lose much of its special usefulness to the Philadelphia authorities.

Philadelphia draws more than 95% of its present enormous supply from five pumping stations on the Schuylkill River, whose water-shed is about 1 900 square miles, and the remainder, or less than 5%, from the Delaware, the water-shed of which, above Philadelphia, is between four and five times as large as that of the Schuylkill.

Between the Hudson, at Albany, and the Delaware, at Philadelphia, there is considerable analogy. Both cities draw from tide-water; each is within a half day of the sea, by water; in each case city

* Capacity allotted, 50 000 000 galls. per day. Rate assumed, 3 000 000 galls. per acre per day.

Mr. Trautwine. sewers discharged into the river both above and below the intake prior to filtration; in each the design of the filtration plant involves the removal of the intake to a point above the discharge of city sewage; and the water-shed areas of the two rivers, above the two cities, respectively, are nearly equal.

Mr. Hazen gives the population per square mile of the Hudson, above Albany, as 33 in 1880, and 43 in 1890, from which we may assume 38 as the corresponding figure for 1885; while Mr. Hering, in his report on extension of water supply in 1885, gives the population of the water-shed of the Delaware, above Philadelphia, as 59 per square mile, including the Lehigh water-shed, and 54 exclusive of that water-shed.

The distribution of the population in each of the two water-sheds is shown graphically in Fig. 21, in which they are compared also with that of the Schuylkill.

All the data of population shown in Fig. 21, are those for the year 1885, they having been found convenient of access in each case. While those of to-day would of course show a marked increase in most cases, yet the changes which have occurred are probably not of such a nature as to interfere with the usefulness of the figures given, for the immediate purpose in hand, viz., an exhibit of the relation between the three rivers in regard to density and distribution of population.

For the Schuylkill and Delaware the populations have been taken from a map prepared by Mr. Rudolph Hering, and published with his report of 1885 on the improvement of the Philadelphia supply. In the case of the Hudson they were found by averaging the figures given by Mr. Hazen, in Table No. 1, for the years 1880 and 1890. The distances, for the Schuylkill and Delaware, were roughly scaled from the map mentioned. Those for the Hudson are given by Mr. Hazen in Table No. 1. In no case has any attempt at accuracy been made.

A striking feature, in the case of the Hudson, is the large population massed upon the banks of the stream at Troy and Watervliet, only 4 miles above Albany, with a further addition of population, half as great, within the next 4 miles, giving a total population (in 1885) of 100 000 within 8 miles of the intake. Nothing like this exists on the Delaware. The portion of Philadelphia lying above the intake has, as a rule, only a rural population, and the experts have recommended the removal of the intake to a point just within the upper city limit. The largest town on the Delaware (Trenton, N. J.) had, in 1885, a population of only about 30 000, or less than half of that of Troy and Watervliet, and it is 30 miles above the city. The entire population represented on the diagram as contiguous to the Delaware, within 50 miles of Philadelphia, does not equal that on the Hudson within 5 miles of Albany. Nevertheless, the density of population

Mr. Trautwine.

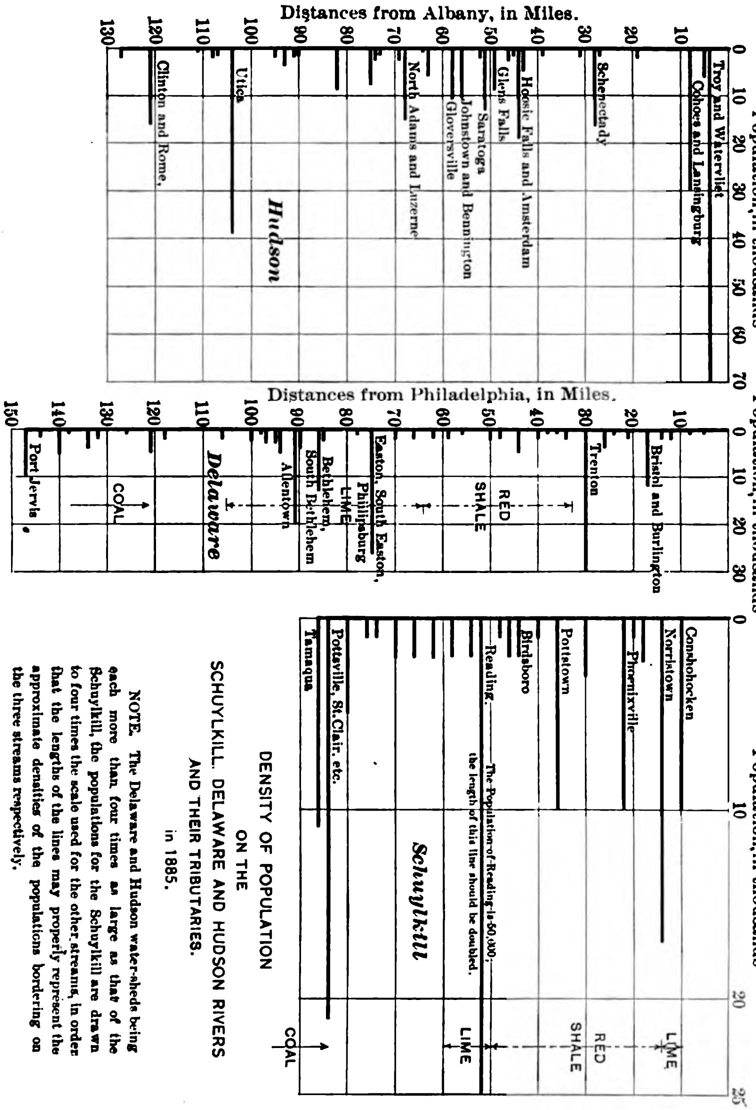


FIG. 21.

Mr. Trautwine. on the entire water-shed of the Delaware, as given by Mr. Hering, for 1885, is more than 50% greater than that for the Hudson, as deduced from Table No. 1.

But while the conditions in the Hudson and the Delaware are thus seen to be somewhat analogous, a glaring contrast appears when we compare either of these two rivers with the Schuylkill; and, inasmuch as Philadelphia now takes more than 95% of her supply from the latter stream, and is to continue to take 75% from it, according to the plan recommended by the experts for immediate execution, the importance of bearing these differences in mind becomes at once apparent.

The water-sheds of the Delaware and the Hudson are each between four and five times as large as that of the Schuylkill, and the density of population is approximately in inverse proportion to the areas. On the map accompanying his report of 1885,* Mr. Hering states the average population of the Schuylkill water-shed above Philadelphia as 176 per square mile, while Mr. Dana C. Barber, in a table accompanying his report of 1884 upon his sanitary survey of the Schuylkill Valley,† gives the area as 1863.9 square miles, and the population as 372 000, making the average density of population 200 per square mile. In order to represent with approximate correctness the relations between the three rivers in the matter of density of population, the writer has plotted the populations for the Schuylkill upon a scale four times as large as for the other two rivers.

Referring to the diagram of the Schuylkill, we find, within about 50 miles of the city, Conshohocken, Phœnixville, Pottstown and Birdsboro, all important iron manufacturing towns, and Norristown, a well-to-do county-seat, once important in the same respect, while, near the head-waters, are Pottsville and Tamaqua, both important anthracite coal mining centers.

Most of these towns are without sewerage systems. Reading is installing such a system, but with commendable consideration for its neighbors down stream, and in the absence of pressure from without, is also installing plants for the filtration of all the sewage led to the river.

Within the city limits of Philadelphia (and, therefore, not shown) and above all but the Roxborough station, is the important textile manufacturing suburb of Manayunk, with a population probably between 10 000 and 15 000. The sewage proper of Manayunk, including most of the discharges from the mills, all of which, until recently, went into the river, is now carried through an intercepting sewer, completed in 1888, to a point below the Fairmount dam, which separates the entire pumpage system from tide water, but much household

* Report of Water Department, Philadelphia, for 1885.

† Report of Water Department, Philadelphia, for 1884.

and other filth is carried directly into the river by storm water, which is not admitted to the intercepting sewer; and probably much more (including, according to official reports, fæcal matter from dwellings of foreign laborers adjacent to the canal of the Schuylkill Navigation Company), is thrown into the canal, from which it passes into the river, barely a mile above the Queen Lane pumping station, the newest and finest of the city's water-works. Mr. Trautwine.

The canals of the Navigation Company, passing, as they do, through most of the towns along the river, and furnishing power to mills there, are naturally made the receptacles of offal, and the nuisance is especially flagrant beyond the city limits, where the city's Board of Health has no jurisdiction. A trip through the canal opposite Norristown, for instance, reveals great accumulations of filth of the most revolting description, scarcely awaiting the next rain to pass into the canal, and thence, in due course, into the river.

The Schuylkill, however, has this advantage over the Delaware and the Hudson, that the two pools, from which the pumps draw their supplies, are at least shut off from tide-water, and thus from the major part of the city's own sewage, by the dam at Fairmount.

Apart from sewage pollution, the Schuylkill suffers mineral pollutions of aggravated character.* In the anthracite regions, at and near its source, it receives sulphuric acid, produced by the oxidation of the iron sulphide occurring in the coal, and large volumes of anthracite coal dust from the washeries, established during recent years for the purpose of extracting the small merchantable sizes from the culm or waste heaps which have been accumulating ever since the opening of the region, and from the "wet breakers," or breakers in which water is used for cleaning the coal and assorting it by sizes. The sulphuric acid is completely neutralized by the limestone of two extensive beds, one just above Reading and the other just above Philadelphia, and the water reaches the city with a basic or "hard" reaction.† The coal dust accumulates in the pools of the Navigation Company, but is swept out and brought down to the city two or three times a year or oftener by floods.

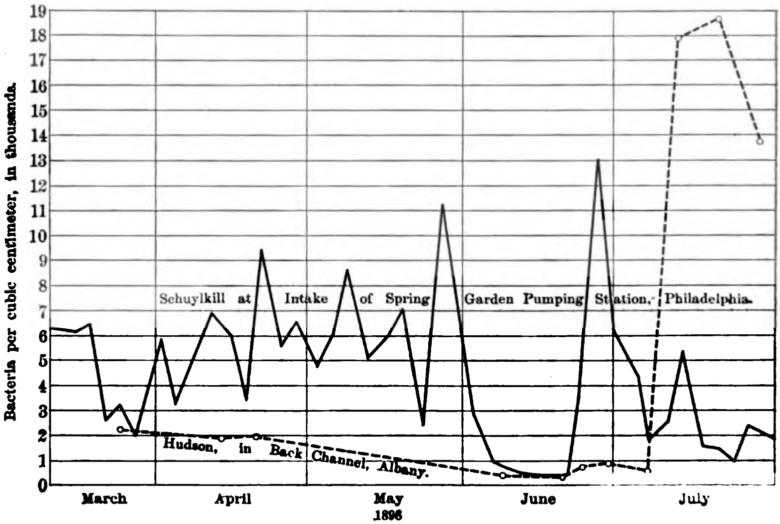
Again, between Reading and Norristown, the Schuylkill passes a broad belt of easily decomposable red shale. In times of flood, this formation sends vast volumes of its substance, in suspension, into the stream, which, at such times, runs blood-red. The first effect of a general storm is, therefore, a visitation of red mud, which, a day or two later, is followed by water charged with coal dust. The writer has seen water drawn from faucets in Philadelphia, after storage in the city reservoirs, so black from coal dust as to be scarcely distinguishable from ink.

* See Fig. 21, in which the limits of the significant geological formations are indicated, both for the Schuylkill and for the Delaware.

† The Delaware water, on the contrary, is quite soft, forming no scale in boilers.

Mr. Trautwine. The Delaware (see Fig. 21) passes through practically these same formations, and receives its modicum of the same adulterants, but, owing to its much larger volume of flow, their effect upon the character of its water is much less marked.

Under the circumstances stated, it is not surprising that Fig. 22



BACTERIA IN SCHUYLKILL AND IN HUDSON RIVER WATERS, UNFILTERED.

FIG. 22.

shows a very much higher average number of bacteria in the Schuylkill water than in that of the Hudson (taken from Table No. 2), until after the "abnormal" conditions following July 9th set in, when the contractors on the Albany work "dumped sand and gravel in the back channel, and took it up again by dredging, for construction purposes, with the result that this water was fouled, and the samples taken after that time do not represent its normal condition."

Unfortunately, the writer is not in position to make a similar comparison between the two rivers in the matter of turbidity. During 1898 (the year for which Mr. Hazen's data in Table No. 2 are given), arrangements had not been completed for using the Hazen scale in observing turbidity in the Schuylkill, and the writer has not succeeded in deriving, from the examinations thus far recorded, a satisfactory coefficient for deducing the readings of the Hazen scale from the record, in parts per million, in which the Philadelphia results for 1898 are stated. The Philadelphia results for March–July, 1899 (measured by the Hazen scale), do not differ greatly from the Hudson results as given by Mr. Hazen; but Mr. George I. Bailey, in his discussion states that the highest turbidity reached since the filters were

put in operation was 0.60; whereas, during the same period, it has on three occasions reached or exceeded that figure in the Schuylkill, as follows: Mr. Trautwine.

August 11th–12th, 1899	0.80
September 27th–29th, 1899	1.50 to 0.60
January 12th, 1900	0.90 to 1.50

In the course of the investigations of the experts, during the summer of 1899, sets of three samples each were taken daily from the Schuylkill and from the Delaware, and examined for (1) the total dry residue contained, (2) the amount of such residue deposited during the first 24 hours, and (3) the amount deposited during the first 48 hours. These observations are still being made, and the Schuylkill results for January 1st–20th, 1900, are indicated in Fig. 23, from which

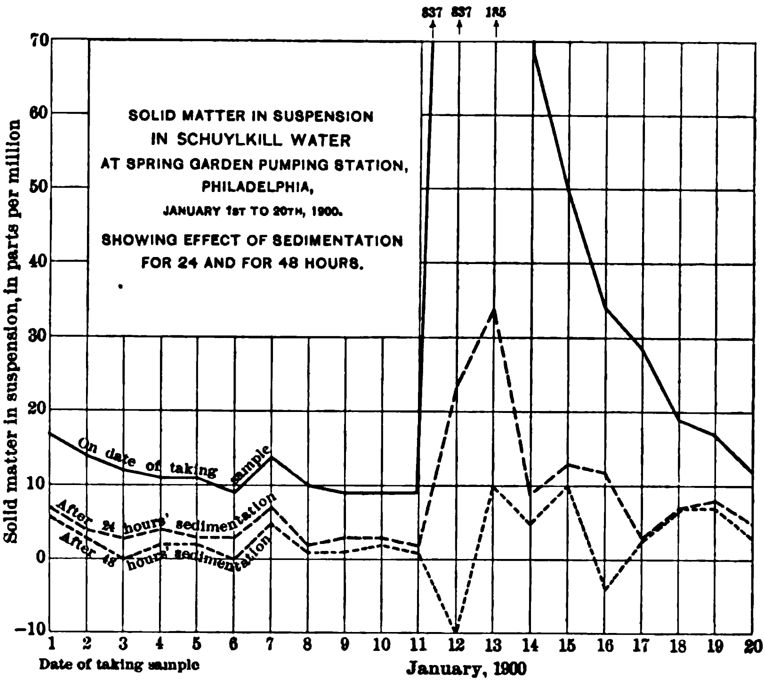
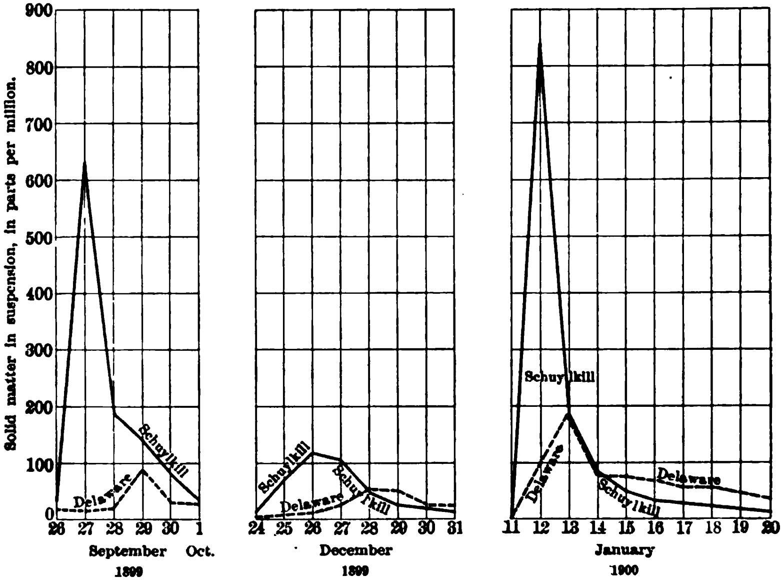


FIG. 23.

appears the startling result that in some cases the amount deposited exceeded the amount originally contained in the water. In others (not shown), the results indicated that less sediment was deposited in 48 than in 24 hours. These erratic results are due, no doubt, to differ-

Mr. Trautwine. ences in the samples as originally taken at approximately one and the same time and place.

Fig. 24 shows a comparison between the Schuylkill and the Dela-



SOLID MATTER IN SUSPENSION IN SCHUYLKILL AND DELAWARE WATERS AT PHILADELPHIA, IN TIMES OF FLOOD.

FIG. 24.

ware as to total solids in suspension during three periods of flood, from which it appears, as might have been expected, that in the smaller stream this effect of floods is not only much more marked and generally of shorter duration, but also appears and disappears earlier than in the larger stream. During normal stages, the sediment in both rivers ranges ordinarily between 10 and 30 parts per million.

Notwithstanding that these investigations indicate in many cases a very nearly complete deposition of all the sediment during the first 24, or at furthest during the first 48, hours, the water, as a matter of fact, often remains visibly turbid for many days thereafter.

These comparisons show that the Schuylkill filters at Philadelphia will not only have a heavier bacterial duty to perform, owing to the greater average number of bacteria in the water, but will also be handicapped by the heavier doses of sediment in its water.

So far as the writer knows, there are on record no data which enable us to judge of the effect, upon the efficiency of the filters, of the coal dust brought down the river at times from the anthracite coal regions.

Mr. Trautwine.

In the writer's judgment, the conditions stated render advisable a high degree of caution in using the construction and operation of the Albany plant as a precedent in the design of the vastly larger system required for the filtration of the quantities of water taken daily from the Schuylkill for the supply of Philadelphia. In taking leave of the city service in November last the writer addressed to the Mayor a communication, urging (as in many previous communications) that one or two of the smaller plants be built first, and "that the construction of filtration plants for the rest of our supply shall proceed as rapidly as we acquire from the two initial plants to be immediately constructed, the knowledge so essential for such an undertaking." It is therefore gratifying to find his Honor recently quoted as saying that, after the plans are completed, which will be "when thorough tests have been made at the new testing station," "it will then be in order to invite proposals for the work of constructing filtration plants at one or more of the city's stations."

With regard to that large portion of its supply which is drawn from the Schuylkill, Philadelphia is fortunate in having at her disposal the large reservoirs heretofore used for storage of the water after pumping, and incidentally for a very imperfect improvement in quality by means of sedimentation. In a report dated February 16th, 1898, to the Director of the Department of Public Works, the writer suggested that the reservoirs be utilized as sedimentation basins, a small portion of their total capacity being set apart for the storage of filtered water, and this feature was an essential one in a system of filtration suggested by the writer on September 9th, 1898, as well as in that recommended by the experts in their report of September 15th, 1899.

As regards the small portion of its supply taken from the Delaware, Philadelphia, like Albany, is without reservoir capacity of a kind which could be utilized conveniently for sedimentation of the water as a preliminary to filtration. Of the two small reservoirs of that supply, the one which has sufficient elevation is small and defective, and has but one basin. Both the experts' and the writer's proposed systems therefore provide for the construction of sedimentation basins for the Delaware supply.

An attempt to compare the cost of construction of the Albany plant, as given by Mr. Hazen in Table No. 3 and in his accompanying remarks, with that of the four proposed slow beds for Philadelphia, as estimated by the experts, is complicated by the difficulty of placing the two systems, with any certainty, upon one and the same basis of comparison. Mr. Hazen says, regarding the Albany plant:

"The filters, sedimentation basin and pure-water reservoir are connected in such a way as to make an exact separation of their costs impossible; but, approximately, the sedimentation basin cost $60 000, the pure-water reservoir $9 000, and the filters $255 000."

Mr. Trautwine. This makes the cost of the sedimentation basin about 18½% of the whole.

Taking the items under "Filters, Sedimentation Basin and Pure-Water Reservoir," in Table No. 3, separating those supposed to be proper to the filter beds and the pure-water reservoir from those common also to the sedimentation basin, using only 81½% of the cost of the latter items, omitting items not included in the Philadelphia experts' estimates, as quoted below, and, finally, deducting $9 000 as the cost of the pure-water reservoir, we arrive at the estimate of the cost of the Albany filter beds, given in Table No. 12.

TABLE No. 12.—Cost of Filter Beds and Appurtenances at Albany.

(*a*) Items common to filters, pure-water reservoir and sedimentation basin.

Preliminary draining	$1 956.71
Excavation	21 761.64
Embankment	8 340.80
Filling	8 360.00
" rolled	3 960.00
Puddle	8 973.25
Concrete in floors	27 112.47
Other concrete	6 703.11
Cement	61 368.52
Extra work and minor items	10 150.11
	$153 686.61
81½ per cent	$125 254.59

(*b*) Items proper to filter beds and pure-water reservoir.

Gravel for lining	$1 508.75
Stone for lining	1 850.74
Concrete in vaulting	29 999 20
Brick work	35 603.75
Filter gravel	7 645.05
Filter sand	36 100.00
Vitrified pipe	7 153.32
Manhole covers	2 956.80
Sand-run fixtures	3 260.00
Regulator houses	6 897.92
Fence	1 704.00
	$259 934.12
Deduct cost of pure-water reservoir	9 000.00
	$250 934.12

or, taking the capacity at 15 000 000 galls. per day, $16 729 per million gallons of daily capacity.

For the four slow plants, at Philadelphia, we have the figures in Table No. 13.

The figures for Philadelphia, in Table No. 13, include excavation for a pure-water basin for Belmont and for Torresdale, and for a sedimentation basin at the latter station, but, from the contours of the sites it would appear that the major portion of the excavation, in each case, was for the filter beds proper. Besides, in the case of the Albany plant, no doubt much more than the 81½% which we have taken for excavation is chargeable to the sedimentation basin.

Much of the very considerable difference between the cost at Albany and the estimates of the experts for Philadelphia may, no doubt, be

Mr. Trautwine.

TABLE No. 13.—ESTIMATED COST OF SLOW FILTER BEDS AND APPURTENANCES, FOR PHILADELPHIA. DEDUCED FROM REPORT OF EXPERTS.

PLANT.	BELMONT.		ROXBOROUGH.		QUEEN LANE.		TORRESDALE.		TOTAL.	
Capacity, in United States gallons per day.	27 000 000.		15 000 000.		58 000 000.		50 000 000.		150 000 000.	
Clearing	$9 000	....	$8 000	....	$10 000	....	$10 000	....	$37 000	....
Excavation	31 200	....	19 249	....	66 382	....	55 383	....	172 214	....
Sand washers*	3 200	....	2 200	....	7 200	....	5 600	....	18 200	....
Drains	1 530	....	1 180	....	7 390	....	6 060	....	16 110	....
Beds	490 568	....	303 647	....	1 009 949	....	904 236	....	2 708 450	....
Fence	15 000	Add 15% †	10 800	Add 15% †	15 600	Add 15% †	5 000	Add 15% †	46 400	Add 15% †
Total	$550 498	$633 073	$345 076	$396 838	$1 116 521	$1 283 999	$986 279	$1 134 221	$2 998 374	$3 448 129
Per million gallons daily	20 389	23 447	23 005	26 456	19 250	22 138	19 725	22 684	19 989	22 987
Excess over Albany ($16 729).	22%	40%	38%	58%	15%	32%	18%	36%	19½%	37%

* Including "piping to sand washers." † 15% added by experts, presumably to cover "engineering and contingencies."

Mr. Trautwine. explained by the advance in prices between the execution of the Albany contracts and the making of the Philadelphia estimates; but, after making all allowances, it would appear that the experts' estimates, at least after their addition of 15%, are certainly safe.

Table No. 14 is a comparison of the total costs of improvement.

TABLE No. 14.—COMPARISON OF THE TOTAL COST OF IMPROVEMENT.

	Philadelphia (estimated).		Albany (actual).
	(a)	(b)	
Population assumed	1 333 333	1 333 333	95 000
Average daily consumption, in gallons per capita	150	75	158
Total gallons	200 000 000	100 000 000	15 000 000
Cost of installation:			
Land	$1 120 353	$100 000	$8 290
Filter plants, piping, sedimentation basins and accessories	6 818 468	3 235 000	323 960
Pumping stations	1 514 770	56 500	49 745
Mains	1 520 000	422 000	86 688
	$10 973 591	$3 823 500	$468 683
Waste restriction	100 000	1 000 000	0
Additions to and improvement of existing plant	3 290 500	1 107 000	0
Engineering and contingencies	Included.	Included.	28 000
Total	$14 364 091	$5 930 500	$496 683
Per million gallons daily	$71 820	$59 305	$33 109

Of the two estimates given for Philadelphia, (*a*) is that of the experts, taken from pages 107, 122 and 123 of their report, and (*b*) is based upon a report by the writer, made September 9th, 1898, to the Director of the Department of Public Works, in response to a resolution of Councils. In order to make estimate (*b*) properly comparable with the other two, it has been amended by adding 50% to the estimated cost of the filters, to provide for their roofing, and an item of $700 000 to provide for roofing the clear-water reservoirs.

The resolution requesting this estimate asked for "plans and drawings and estimated cost of filtration of all the water used by the city," and the writer took advantage of the word "used," to show the city fathers what could be done, in the way of proper economy, by restricting the consumption to a figure (maximum 100 galls. per capita per day), more nearly commensurate with the quantity (possibly 50 galls. per day, maximum) really used.* The estimate was based upon the Albany bids, which then had just been opened, and the amounts of which were kindly communicated to the writer by Mr. Hazen for the purpose of the estimate.

* The result was a resolution, by the Water Committee, requesting the Department "to place in proper form the results of its researches on the question of Slow Sand Filtration for the entire city."

The figures indicate an economy out of proportion to the proposed reduction of consumption, but this was to be expected. For the smaller quantity, sites were generally available on city property adjoining or near to existing plants, whereas, for the larger quantity, sites for the filter beds had, in all cases, to be acquired. In other ways, also, and notably in the matter of mains to connect existing works with filter beds perforce located at considerable distance, the total cost, in Philadelphia, increases much more rapidly than the quantity to be filtered. Mr. Trautwine.

Philadelphia lies just south of the line which Mr. Hazen has drawn to indicate at what locations it is advisable to roof over the filter beds, and the question of the advisability of doing so for Philadelphia may be one for the authorities to discuss. Poughkeepsie, after some twenty years' experience with an uncovered bed, built another, and the writer, largely on the strength of this, omitted roofing, in designing for Philadelphia the system already mentioned, but he has since been informed that the building of a second uncovered filter bed at Poughkeepsie was contrary to the advice of Mr. Fowler, the Superintendent.

As to cost of operation, the experts make estimate for Philadelphia, as follows:

"Per million gallons of filtered water, including labor, cost of wash and waste water, lost sand, sanitary analyses of water, chemicals, superintendence, watchmen, ordinary repairs, and all incidental expenses; but excluding interest, depreciation and cost of pumping water to filters:

	Schuylkill River.	Delaware River.
Slow filters......................	$3.60	$3.00
Rapid filters.....................	4.80	4.00

"Cost of pumping, per million gallons raised 1 ft. high, including coal, labor, oil, waste and supplies, and ordinary repairs; but excluding interest and depreciation:

"Low-Lift Pumps.

"For a daily supply of 200 000 000 galls., 5.25 cents."

In his report of September 15th, 1898, the writer, with much misgiving, due to insufficiency of data, ventured an estimate of $3.97 per million gallons as the cost of slow filtration proper, and $0.41 per million gallons as the cost of raising the water by low-lift pumps, making a total of $4.38.

Considerable research, including a correspondence with all the American filtration plants of which the writer could learn, developed costs of filtration alone, ranging all the way from $1.50 to $10 per million gallons filtered, the latter figure represented by Lawrence, Mass., and the writer concluded that:

"In the absence of more exact information, and in view of the excessive turbidity of our water in flood and of the rate of wages fixed by ordinance of Councils for an 8-hour day's labor, it would be unsafe to estimate the cost of operation (exclusive of interest on cost), for the filters contemplated, at less than $3 per million gallons, notwithstand-

Mr. Trautwine. ing the large dimensions of the proposed works which should conduce to economy, and notwithstanding that the proposed reduction in consumption would greatly facilitate sedimentation. On the other hand, with proper economy, the cost could not exceed $5."

Mr. Bailey's experience with the Albany filters, as deduced from their operation from September 5th to December 25th, 1899, inclusive, and stated in his discussion, shows a cost, for filtration proper and laboratory work, of $1.67, and $2.52 for raising the water from the river to the sedimentation basin, making the total cost $4.19 per million gallons.

Mr. Hazen. ALLEN HAZEN, M. Am. Soc. C. E. (by letter).—The discussion has brought out a large number of practical points, both in reference to the Albany plant and to filtration in general.

The results of operation given by Mr. Bailey are most gratifying, and the cost of operation, as exhibited in the comparison made by Dr. Mason, is very favorable, and shows an excellent organization of the work.

The water quantities given by Mr. Bailey are taken directly from the filter records. Some preliminary experiments have indicated that the coefficient of discharge assumed in computing the orifices was too small by about 5%, and that this amount should be added to the results. The value of the coefficient of discharge seems to be almost exactly the same whether the orifice is submerged or not. It is the intention to make more precise determinations, and afterward to have new and correct scales painted and substituted for those now in use.

Since the presentation of the paper a flood has occurred higher than any on record except that of 1857, which was a little higher. No damage was caused, but when the water exceeded the height of the overflow in the sedimentation basin, the river water entered it in that way. One of the pumps was kept in operation at a low rate to keep the pumping station dry. The operation of the filters was not interrupted or interfered with in any way.

In discussing the vaulting, Mr. Hill has made certain computations of quantities, and in doing so he has, apparently, divided the total amounts of concrete in the vaulting and in the flooring by the number of bays. In doing this, the writer thinks, he has overlooked the number of bays in the pure-water reservoir, and has also overlooked the fact that nearly half of the concrete in the floors was in the sedimentation basin and had nothing to do with the filters. He has also omitted the cost of the cement.

The figures for one section, 13 ft. 8 ins. square, corrected, and adding the price of the cement, are approximately as follows:

As executed:

5.4 cu. yds. of vaulting, at $6.30	$34.02
4.85 cu. yds. of flooring, at $4.75	23.04
1.24 cu. yds. of brick work, at $9.67	11.99
	$69.05

Mr. Hazen.

As proposed:

10 cu. yds. of roof slab, supporting column, floor and foundation, at $4.75....	$47.50
Centering, 4 cents per square foot....	7.47
Expanded metal....	10.60
	$65.57

The 5.4 cu. yds. of concrete vaulting, per section, given above, includes the proportionate part of all special structures, and of the excess weight of the cylindrical vaulting near and over the walls. Without these, but including the manholes, the actual amount of concrete was only 5.1 cu. yds. per section.

The exact form of construction suggested by Mr. Hill was not considered, but several others of the same general type were studied before the plans were put in final shape. Some of these methods appeared very promising. The cheapening, however, depended upon a reduction of the thickness of concrete to less than 6 ins. While good results might no doubt have been obtained with concrete 3 or 4 ins. thick, reinforced with steel, there was no known precedent for its use under similar conditions. The Water Board was unalterably opposed to the use of any form of construction which could be regarded as in any degree experimental, and for this reason it was decided to use only types of construction which were well demonstrated. It was therefore necessary to postpone until another time a practical trial of the steel and concrete construction.

It would have been better to have covered the whole of the vaulting with a thin layer of sand before placing the silt and soil, and, particularly, to have surrounded the manholes with gravel. The draining would have been facilitated by this procedure, and the lifting of the manholes by frost would have been made impossible.

The estimate of the cost of vaulting per square foot, given by Mr. William B. Fuller, is a little greater than that given by the author, recently, in discussing Mr. Metcalf's paper.* The difference arises from, first, the fact that Mr. Fuller has reckoned the cost of the vaulting upon the net filtering area, while the author reckoned it upon the whole area covered; and, second, the fact that Mr. Fuller has included in the cost of the vaulting the cost of that part of the floor which he assumes to be due to its use as a foundation. The cost of vaulting, of course, is much the largest element of difference between the costs of open and covered filters, but it should be remembered that there are other points of difference, and that deducting the cost of vaulting from the cost of covered filters does not necessarily give the cost of open filters. Correct comparisons can only be made by examining corresponding designs for filters of the two types, using the same unit prices.

* *Transactions*, Am. Soc. C. E., Vol. xliii, p. 68.

Mr. Hazen. Mr. William B. Fuller and Mr. Fowler have suggested that covered filters have other advantages than protection against frost which may make their construction desirable, even in climates where covers are not necessary to prevent ice. This may sometimes be the case, but it should be borne in mind that, aside from the question of ice, there are distinct advantages and disadvantages arising from the use of covers. The writer does not propose to discuss this question at length; but to show that this opinion is not unanimous he will state that Dr. Strohmeyer, after making the extended investigations mentioned in Mr. Whipple's discussion, states that he has come to the conclusion that open filters have decided advantages over covered ones, quite aside from considerations of difference in cost. Mr. Trautwine also raises this question. Of course, in severe climates there is no question as to the necessity of covering filters. It is only where the winters are not too severe that the question arises.

The sufficiency of the vaulting as a protection against cold has been tested during the past winter. Ice has formed to a thickness of 3 or 4 ins. immediately about the entrances to the filters, but it has been found possible to break up this ice and let it pass through the gates leading to the overflow chambers as the water is drawn from the filters before cleaning. Over the rest of the filters a skim of ice has formed occasionally, but this could be thrown aside during cleaning, and has not seriously interfered with the work.

Mr. Rafter is right in stating that the writer assumed that, on the whole, cracks are to be expected. It is a matter of common experience that small masonry structures remain entirely free from cracks. As the size of the structure increases, particularly if the masonry is comparatively light, the probability of cracks increases. It should be remembered that temperature contraction is only one of the causes of cracks in masonry. Cracks often occur through settlement, and at Albany six cracks were caused by the lifting of the walls by frost, due to the exposure of some of the work in an uncompleted condition. When a crack has once occurred it is not an easy matter to repair it so that the wall will be as strong as it was originally, and it then makes little difference whether it was caused originally by settlement, temperature or other causes.

When the plans were being drawn, the question of the bearing power of the foundations was considered quite seriously. The site was a soft marsh. Borings showed a hard material at a comparatively slight depth over the greater part of the area. When test pits were dug, a clay was found fairly hard as first exposed, but shrinking considerably on drying, and, if disturbed in contact with water, becoming very soft. Experiments were made by loading 1 sq. ft. of this material, and some settlements were observed. A part of the filters was to rest upon this foundation, a part upon rock, and a small part upon

still softer clay. It was expected that there might be settlement in parts of the work. With this in view, an attempt was made, first, to distribute the weight over as large an area as possible, thereby reducing the probability of settlement; second, to load the whole foundation as evenly as possible, so that in case settlement occurred the whole structure would go down together with the minimum damage; and, third, the possibility of settlement and of cracks was contemplated in designing all parts of the structures, and the endeavor was made to arrange the masonry so that it would suffer as little as possible in case of settlement. Provision for cutting off water coming through cracks was also made, as described in the paper. Mr. Hazen.

When the excavation was made, the foundation proved to be much better than we had felt safe to assume from the character of the borings and test pits; and the fact that no measurable settlement occurred on any part of the filters must be attributed rather to the natural excellence of the foundations than to the design. In the pumping station some settlement actually took place, but the cracks resulting from it have not proved serious.

The cross-walls were built of brick instead of concrete, principally because the writer supposed that brick would be less likely to crack than concrete; and also because he believed that in case it did crack, the cracks could be repaired more easily. The writer's more recent experience with concrete, in this respect, has been quite favorable; and in a smaller plant, since designed and built, at West Superior, Wis., all the walls are of concrete. The bonding in these concrete walls was made as described in the paper, and the writer believes that in this way the joints between the old and new work are made substantially as strong as any part of the work. When cracks occurred at Albany, they went straight through the walls, and did not follow any of these joints.

To build masonry structures without cracks is certainly desirable. The writer believes, however, that a much larger proportion of the inferior work performed by filtration plants than is generally supposed has resulted from cracks in the masonry structures; and until the art of masonry construction is so far advanced as to make it quite certain that structures can be built without cracks, it is justifiable and necessary in designing filters, to consider the probable effect of cracks, and to take precautions against the damage which might result from them. Such precautions can be made entirely effective, and at a comparatively small expense. The writer believes that the design at Albany is such that, had there been many more cracks, or had there been some little settlement in portions of the work, the structures would still have maintained their stability, and would have continued to serve the purpose for which they were built.

In reply to Mr. Maignen's question, the sand-washing apparatus

Mr. Hazen. is not protected from the weather, and no sand is washed during the winter months. The dirty sand during this time is piled up in the court to be washed in the spring. It would, no doubt, be possible to protect the apparatus so as to wash in winter, but there is no objection to allowing the sand to remain until spring.

In regard to the thickness of the gravel layers, mentioned by Mr. Fowler and Mr. G. W. Fuller, the writer has constructed and observed many filters during the last dozen years, many of them experimental, and a smaller number for actual work. In many cases he has used gravel layers thinner than those at Albany. He does not remember that he has ever used thicker layers in the aggregate, although in some cases the minimum over the tops of the drains may have been a little greater. He has had occasion to examine quite a number of the drains after they had been in use for considerable periods, and in no case has he been led to think that thicker gravel layers would have been desirable. Thin gravel layers, of the right sizes and carefully placed, are, in his opinion, quite as good as thicker ones. Much thicker layers, carelessly placed, or not of the right sizes, are entirely inadequate. It is cheaper and better in every way to use comparatively thin layers, and to take the trouble to make them right, and have them effective, than to use thicker layers and depend to a certain extent upon chance for the results.

The methods adopted by the contractors at Albany for screening and placing the gravel were particularly satisfactory. There has not been the slightest evidence of incomplete support, nor does the writer consider that there is any danger that the sand will get into the gravel so as to injure the filters.

In this connection, it may be stated that the coarse gravel over the drains, and particularly about the joints, was placed with unusual care. The workmen were instructed personally in the methods to be adopted in this part of the work. They quickly became skillful at it, and were faithful in carrying out their instructions.

Mr. Fowler mentions the clogging of the holes in the inlet pipes to the sedimentation basin. In practice, it is found that these holes are stopped up, to a certain extent. The appearance of the outlets is not very much changed until a considerable proportion of the holes is stopped. Practically, the rate of pumping always exceeds the capacity of the holes, and water is always flowing over the tops of the pipes. It is necessary to clean the holes at intervals, and this is done with a broom, by a man in a boat. The cleaning is done very quickly. It can be done at any time when the pumps are stopped, or, when they are running, by shutting off the inlets in rotation. The inlets have actually been cleaned at intervals of a week or two, and the labor required was very slight.

At another time, the writer would put the inlets somewhat farther

away from the bank. With this done, it would be possible to leave the perforated pipes in position during the winter. As it is, the spray from them builds ice upon the bank. They were actually taken off soon after the commencement of cold weather. This is not a very important matter, as they are easily taken off; there is probably no necessity for aeration during the winter, and, with the lower lift, a little coal is saved. Mr. Hazen.

The question raised by Mr. Fowler as to the size of individual filters and the number of beds in a plant, is an important one. It should be remembered, however, that while the convenience of operation is generally increased by increasing the number of beds, the cost of construction per unit of area is also increased, and it may often be found best to sacrifice something of present convenience to economy of construction; or, in other words, with a given sum of money, to build a larger area of large beds, rather than a smaller area of small ones.

The comparisons of populations upon several rivers, mentioned by Mr. Trautwine, are extremely interesting. It should be noted, however, that the figures given by the writer for the Hudson River include only the populations of cities, towns and villages, and, as stated in the paper, do not include the rural population. The rural population, obtained by going over the water-shed by counties, and deducting the populations of cities, towns and villages given in the table, amounts to about 30 per square mile for 1890. The rural population does not change rapidly, and a close approximation to the total population at the various dates would be obtained by adding 30 per square mile to the figures for the urban population given in the table.

The methods of distribution of the cost of various parts of the plant, given by Mr. Trautwine, are not entirely clear to the writer. The figures given in the paper were obtained by going over each item somewhat carefully, and allotting the proper proportion to the structures indicated. The final results were given in round numbers, because of the impossibility of deciding the exact points of division between the various parts mentioned.

The question raised by Mr. G. W. Fuller, as to the quality of the raw water, has been already answered in part by Mr. Bailey in his discussion. Data upon these points are rapidly being accumulated at the laboratory of the works, and much more complete results will be available after a little while. The data at hand at the time of the preparation of the paper were so meager in comparison with those now being obtained, that it was thought best to let the matter stand until it could be discussed more adequately.

The general character of the water, as regards muddiness, is between the very clear waters of most New England streams, and the turbid waters of the Middle States, although it resembles the former

Mr. Hazen. more than the latter. Generally, the raw water is comparatively clear, but very muddy water is obtained occasionally, especially from the Mohawk, which drains a clayey country from which the water is quite muddy. The fluctuations in turbidity are much less rapid at Albany than in some other places, and when the water becomes turbid it remains so for several days. The greatest turbidity yet observed is such that a bright platinum wire 0.04 in. in diameter can be seen through only 1 in. of water.

In reply to Mr. Whipple's question, Bleecker Reservoir, which is perhaps the most important of the distributing reservoirs, has recently been thoroughly cleaned. The laboratory in connection with the filter plant will make studies of the vegetable growths which occur in the reservoirs, but it is too early to draw any conclusions upon this point.

The odor due to gas waste, mentioned by Dr. Mason, first occurred after the paper was prepared. The odor was offensive on only one occasion during the fall, resulting from a combination of extremely low water, spring tides, and a south wind. Dr. Mason has indicated the proper solution of this problem, and the gas company has made arrangements to build a drain to the Patroons Creek sewer, so that this material, instead of being discharged into the back channel, will be discharged into the river below the southern end of the island, where the opportunities for dilution will be much greater. It is also to be hoped that improved methods at the gas works will reduce the amount of this highly objectionable material to be discharged into the river. It should be understood that this odor was due to material discharged into the back channel at a point near the intake, and carried up to it in a comparatively concentrated condition, and not to material which had become mixed with a considerable portion of water flowing in the river.

Vol. XXVI. MAY, 1900. No. 5.

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Edited by the Secretary, under the direction of the Committee on Publications.

CONTENTS.

SOCIETY AFFAIRS.

PAPERS AND DISCUSSIONS.

ILLUSTRATIONS.

The prices of publications are as follows: *Proceedings*, $6 per annum; *Transactions*, $10 per annum. Postage will be added when *Proceedings* are sent to foreign countries.

American Society of Civil Engineers.

OFFICERS FOR 1900.

President, JOHN FINDLEY WALLACE.

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ROBERT MOORE.	ALFRED NOBLE.

Secretary, CHARLES WARREN HUNT.

Treasurer, JOSEPH M. KNAP.

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Term expires January, 1901:	*Term expires January, 1902:*	*Term expires January, 1903:*
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JOHN KENNEDY,	C. W. BUCHHOLZ,	HENRY B. SEAMAN,
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Assistant Secretary, T. J. MCMINN.

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Special Committees.

ON ANALYSIS OF IRON AND STEEL:—Sub-Committee of the American Society of Civil Engineers (of the International Committee on Standards for the Analysis of Iron and Steel, of which Prof. J. W. Langley is Chairman)—Charles B. Dudley, William Metcalf, Thomas Rodd.

ON UNITS OF MEASUREMENT:—George M. Bond, William M. Black, R. E. McMath, Charles B. Dudley, Alexander C. Humphreys.

ON THE PROPER MANIPULATION OF TESTS OF CEMENT:—George F. Swain, Alfred Noble, George S. Webster, W. B. W. Howe, Louis C. Sabin, H. W. York.

The House of the Society is open every day, except Sunday, from 9 A.M. to 10 P.M.

HOUSE OF THE SOCIETY—220 WEST FIFTY-SEVENTH STREET, NEW YORK.

TELEPHONE NUMBER, - - 588 Columbus.
CABLE ADDRESS, - "Ceas, New York."

Vol. XXVI. MAY, 1900. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PROCEEDINGS.

SOCIETY AFFAIRS.

CONTENTS:

MINUTES OF MEETINGS.

OF THE SOCIETY.

May 2d, 1900.—The meeting was called to order at 8.45 P. M., Vice-President Rudolph Hering in the chair; Charles Warren Hunt, Secretary, and present, also, 73 members and 14 visitors.

The minutes of the meetings of April 4th and 18th, 1900, as printed in *Proceedings* for April, 1900, were approved.

A paper by J. M. Moncrieff, M. Am. Soc. C. E., entitled "The Practical Column under Central and Eccentric Loads," was presented by the Secretary, who also presented a written discussion on the paper by Ernst F. Jonson, Assoc. M. Am. Soc. C. E. The subject was discussed orally by Messrs. W. H. Breithaupt, C. T. Purdy and H. S. Prichard.

Ballots were canvassed and the following candidates declared elected:

As Members.

Robert Adam, Vera Cruz, Mex.
Frank Valentine Erhard Bardol, Buffalo, N. Y.
Robert James Beach, Brooklyn, N. Y.
John Benjamin Body, Coatzacoalcos, Mex.
Alfred Willard French, Brooklyn, N. Y.
Clifford Stephen Kelsey, Westfield, N. J.
James Henry Kennedy, St. Thomas, Ont.
John Langton, New York City.
Charles Oscar McComb, Watertown, N. Y.
William Moncure, Raleigh, N. C.
John Austin Patterson, Philadelphia, Pa.
Alexander Bell Ross, Leavenworth, Kans.
Clarence Browning Vorce, Hartford, Conn.
Mark Albigense Waldo, Leavenworth, Kans.

As Associate Members.

William Russell Davis, Albany, N. Y.
Lawrence Griffith, New York City.
Charles Rufus Harte, Jamaica Plain, Mass.
John Bruce Hayden, New York City.
Clifton Stewart Humphreys, Madison, Me.
Tillinghast L'Hommedieu Huston, Havana, Cuba.
Robert Edward Moss, Bloomfield, N. J.
Charles Henry Nichols, New Haven, Conn.
Macy Stanton Pope, East Machias, Me.
Leidy Rudy Shellenberger, Philadelphia, Pa.
Henry Hudson Shepard, New Berlin, N. Y.
Charles Winslow Sherman, Boston, Mass.

Announcement was made that the following candidates were elected by the Board of Direction, May 1st, 1900:

As Associates.

Ernest Robinson Ackerman, New York City.
Jacob Herbert Sawyer, Chestnut Hill, Mass.

As Juniors.

Raymond Edmond Adams, Philadelphia, Pa.
Walter Hinds Allen, Durango, Mex.
Eugene Bradford Bumsted, Jersey City, N. J.

ARTHUR MORTIMER DAY, New York City.
HORACE DE REMER HAIGHT, Detroit, Mich.
EDWARD EASTMAN MINOR, New Haven, Conn.
LAZARUS WHITE, New York City.

Adjourned.

May 16th, 1900.—The meeting was called to order at 8.45 P. M., Samuel Whinery, Director, in the chair; Charles Warren Hunt, Secretary, and present, also, 110 members and 24 visitors.

An informal discussion on Structural Steel for Buildings; Wrought Iron; and Structural Steel for Bridges and Ships was opened by Mansfield Merriman, M. Am. Soc. C. E. The subject was discussed by Messrs. H. M. Howe, William R. Webster, Henry Goldmark, H. H. Campbell, W. H. Burr, R. S. Buck and Henry B. Seaman.

The Secretary presented a written discussion on the subject by J. E. Greiner, M. Am. Soc. C. E.

The Secretary announced the death of FRANK PAUL DAVIS; elected Member February 1st, 1888; died May 3d, 1900.

Adjourned.

OF THE BOARD OF DIRECTION.

(Abstract.)

May 1st, 1900.—The Board met at 8.20 P.M., Vice-President Hering in the Chair; Charles Warren Hunt, Secretary, and present, also, Messrs. Buchholz, Deyo, Knap, Manley, Morison, Noble, O'Rourke, Ricketts, Seaman, Turner and Whinery.

Action was taken in regard to members in arrears for dues.

An agreement was authorized reducing the rate of interest on the mortgage on the Society property from 4½ to 4 per cent.

The Secretary was authorized to send a circular showing the geographical districts into which the Society is divided for the purposes of the Nominating Committee, and giving members in each district an opportunity to nominate a representative on that Committee, such nominations to be presented to the Annual Convention.

Applications were considered and other routine business transacted.

Two candidates for Associate and seven for Junior were elected.*

Adjourned.

* See page 142.

ANNOUNCEMENTS.

In accordance with the resolution of the Board of Direction the House of the Society is open every day, except Sunday, from 9 A. M. to 10 P. M.

MEETINGS.

Wednesday, June 6th, 1900.—This will be the last meeting before the summer vacation. Ballots for membership will be canvassed, and a paper by F. A. Kummer, Jun. Am. Soc. C. E., entitled, "A Proposed Method for the Preservation of Timber," will be presented for discussion. This paper is printed in the current number of *Proceedings.*

Wednesday, September 5th, 1900.—A regular business meeting will be held. Ballots for membership will be canvassed, and a paper by Elwood Mead, M. Am. Soc. C. E., entitled, "Irrigation Studies," will be presented for discussion. This paper is printed in the current number of *Proceedings.*

THIRTY-SECOND ANNUAL CONVENTION.

The Thirty-second Annual Convention of the Society will be held at the House of the Institution of Civil Engineers, Great George St., Westminster, S. W., London, England, beginning on Monday, July 2d, 1900, and continuing during that week.

The general arrangements for the Convention are in the hands of a Committee of the Board of Direction, consisting of the following:

Palmer C. Ricketts,
H. S. Haines,
Henry Manley,
Rudolph Hering,
Charles Warren Hunt.

The following Local Committee of Arrangements has been appointed by the Board of Direction:

Sir Benjamin Baker,
Sir Douglas Fox,
Rawlinson T. Bayliss,
James R. Bell,
William Henry Booth,
George Earl Church,
A. G. Glasgow,
Robert Gordon.
Alfred Francis Harley,
John A. McDonald,
Hiram S. Maxim,
William B. Myers-Beswick,
John P. O'Donnell,
M. E. Yeatman,
Albert J. Campbell,
Millard Hunsiker,
J. R. Furman,
Victor M. Clement.

The subjects for informal discussion have been printed in *Proceedings.**

The opening session of the Convention will be held during the afternoon of **Monday, July 2d, 1900,** when the Society will be welcomed by the President of the Institution, *Sir* Douglas Fox; and John F. Wallace, President, Am. Soc. C. E., will deliver the Annual Address.

On Monday evening the first of the topical discussions, relating to High Buildings, will be opened by Corydon T. Purdy, M. Am. Soc. C. E., who will illustrate his remarks with stereopticon views.

Other meetings will be arranged for the discussion of the other subjects, and a time will be set for a Business Meeting. The dates and other particulars of these meetings, as well as excursions to points of interest, etc., will be arranged by the Local Committee.

Members who are unable to attend are invited to send written communications on any of the subjects, for presentation at the meeting.

On the evening of Thursday, July 5th, 1900, the Institution will hold its Annual Conversazione in the ancient Guildhall of the City of London, and has invited the members of the Society, as well as the ladies of their families, to this function.

NOMINATING COMMITTEE.

A map showing the seven geographical districts into which the territory occupied by the membership is divided, for the purposes of the Nominating Committee, is printed on page 146.

The members of the Nominating Committee holding over and the districts they represent are given in the following list:

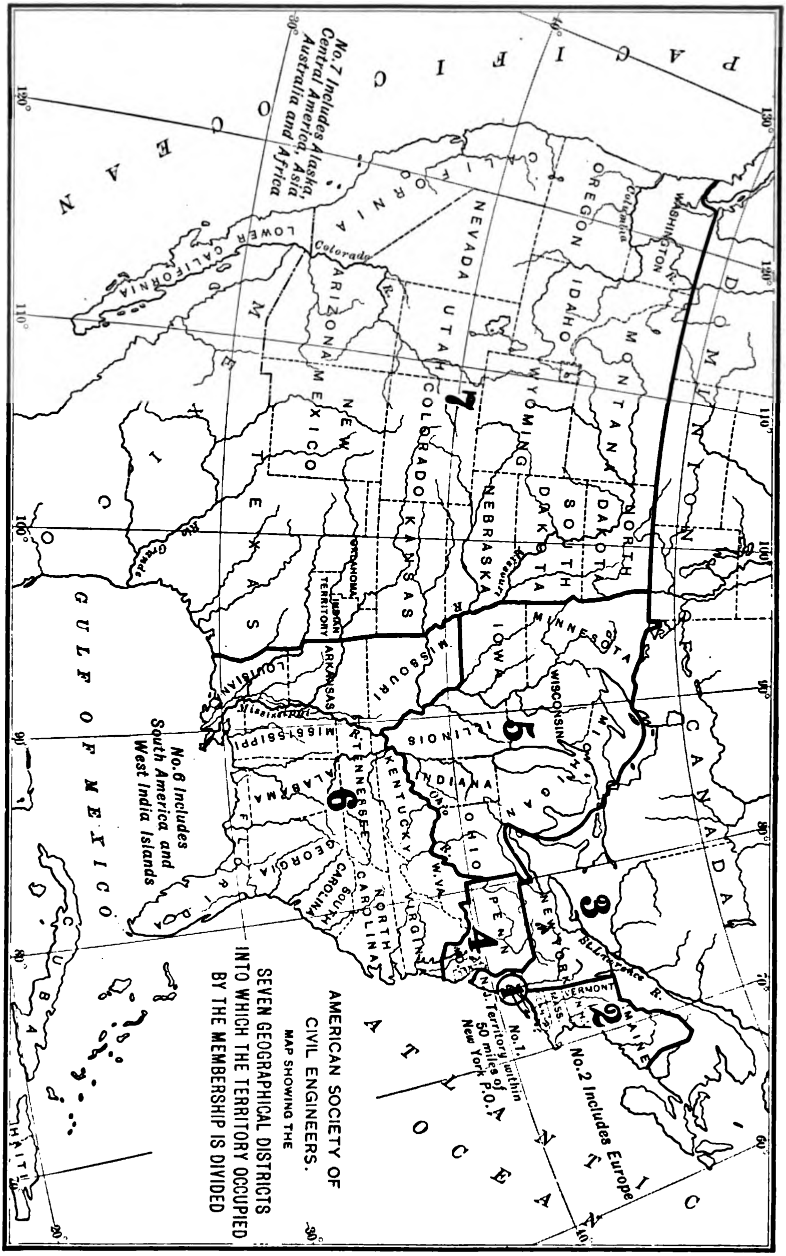
Alfred W. Trotter, District No. 1.	W. G. Wilkins, District No. 4.
George B. Francis, " " 2.	Augustus Mordecai, " " 5.
T. McC. Leutzé, " " 3.	J. A. Ockerson, " " 6.

Hiram M. Savage, District No. 7.

From each district another representative is to be appointed by the Business Meeting of the Annual Convention.

Corporate Members have recently been given the opportunity of sending in the name of their choice, the nominations so made to be presented to the Business Meeting.

* Vol. xxvi, p. 97 (April, 1900).


AMERICAN SOCIETY OF
CIVIL ENGINEERS.
MAP SHOWING THE
SEVEN GEOGRAPHICAL DISTRICTS
INTO WHICH THE TERRITORY OCCUPIED
BY THE MEMBERSHIP IS DIVIDED
No. 1. Territory within 50 miles of New York P.O.
No. 2 Includes Europe
No. 6 Includes South America and West India Islands
No. 7 Includes Alaska, Central America, Asia, Australia and Africa
PACIFIC OCEAN
ATLANTIC OCEAN
GULF OF MEXICO
DOMINION OF CANADA
MEXICO
LOWER CALIFORNIA
CUBA
HAITI

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST.

(April 11th to May 9th, 1900.)

NOTE.—*This list is published for the purpose of placing before the members of the Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS.

In the subjoined list of articles references are given by the number prefixed to each journal in this list.

(1) *Journal*, Assoc. Eng. Soc., 257 South Fourth St., Philadelphia, Pa., 30c.
(2) *Proceedings*, Eng. Club of Phila., 1122 Girard St., Philadelphia, Pa.
(3) *Journal*, Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal*, Western Soc. of Eng., Monadnock Block, Chicago, Ill.
(5) *Transactions*, Can. Soc. C. E., Montreal, Que., Can.
(6) *School of Mines Quarterly*, Columbia Univ., New York City, 50c.
(7) *Technology Quarterly*, Mass. Inst. Tech., Boston, Mass., 75c.
(8) *Stevens Institute Indicator*, Stevens Institute, Hoboken, N. J., 50c.
(9) *Engineering Magazine*, New York City, 30c.
(10) *Cassier's Magazine*, New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, New York City, 35c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News*, New York City, 15c
(14) *The Engineering Record*, New York City, 12c
(15) *Railroad Gazette*, New York City, 10c.
(16) *Engineering and Mining Journal*, New York City. 15c.
(17) *Street Railway Journal*, New York City, 35c.
(18) *Railway and Engineering Review*, Chicago, Ill.
(19) *Scientific American Supplement*, New York City, 10c.
(20) *Iron Age*, New York City, 10c.
(21) *Railway Engineer*, London, England.
(22) *Iron and Coal Trades Review*, London, England.
(23) *Bulletin*, American Iron and Steel Assoc., Philadelphia, Pa.
(24) *American Gaslight Journal*, New York City, 10c.
(25) *American Engineer*, New York City, 20c.
(26) *Electrical Review*, London, England.
(27) *Electrical World and Electrical Engineer*, New York City, 10c.
(28) *Industries and Iron*, London, England.
(29) *Journal*, Society of Arts, London, England.
(30) *Annales des Travaux Publics de Belgique*, Brussels, Belgium.
(31) *Annales de l' Assoc. des Ing. Sortis des Ecôle Spéciales de Gand*, Brussels. Belgium.
(32) *Memoirs et Compt Rendu des Travaux*, Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil*, Paris, France.
(34) *Portefeuille Economique des Machines*, Paris, France.
(35) *Nouvelles Annales de la Construction*, Paris, France.
(36) *La Revue Technique*, Paris, France.
(37) *Revue de Mecanique*, Paris, France.
(38) *Revue Générale des Chemins de Fer et des Tramways*, Paris, France.
(39) *Railway Master Mechanic*, Chicago, Ill.
(40) *Railway Age*, Chicago, Ill., 10c.
(41) *Modern Machinery*, Chicago, Ill., 10c.
(42) *Transactions*, Am. Inst. Elec. Eng., New York City, 50c.
(43) *Annales des Ponts et Chaussées*, Paris, France.
(44) *Journal*, Military Service Institution, Governor's Island, New York Harbor, 75c.
(45) *Mines and Minerals*, Scranton, Pa. 20c.
(46) *Scientific American*, New York City, 10c.
(47) *Mechanical Engineer*, Manchester, England.
(48) *Zeitschrift des Vereines Deutscher Ingenieure*, Berlin, Germany.
(49) *Zeitschrift für Bauwesen*, Berlin, Germany.
(50) *Stahl und Eisen*, Duesseldorf, Germany.
(51) *Deutsche Bauzeitung*, Berlin, Germany.
(52) *Rigasche Industrie-Zeitung*, Riga, Russia.
(53) *Zeitschrift des oesterreichischen Ingenieur und Architekten Vereines*, Vienna, Austria.
(54) *Den Tekniske Forenings Tidsskrift*, Copenhagen, Denmark.
(55) *Ingeniören*, Copenhagen, Denmark.
(56) *Teknisk Tidskrift*. Stockholm, Sweden.
(57) *Teknisk Ugeblad*, Christiania, Norway.
(58) *Proceedings*, Eng. Soc. W. Pa. 410 Penn Ave., Pittsburg, Pa. 50c.
(59) *Transactions*, Mining Institute of Scotland, London and Newcastle-upon-Tyne.
(61) *Proceedings*, Western Railway Club. 225 Dearborn St., Chicago, Ill., 25c.
(62) *American Manufacturer and Iron World*, 59 Ninth St., Pittsburg, Pa.
(63) *Minutes of Proceedings*, Inst. C. E., London, England.

LIST OF ARTICLES.

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The Position of Wheel Loads Causing Maximum Stresses in Web Members.* M. A. Howe. (15) April 20.
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The Design of Rotary Converters.* H. F. Parshall and H. M. Hobart. (11) Serial beginning Sept. 29, 1899, ending April 27, 1900.
The Willesden Electricity Works of the Metropolitan Electric Supply Company, Limited.* (26) Serial beginning March 23, ending April 6.
The Electric Wiring of Buildings.* W. A. Chamen. (47) Serial beginning March 31, ending April 14.
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New Iron and Steel Works Plant.* (22) Serial beginning March 9, ending April 13.
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Canal de la Marne à la Saône. Gustave Cadart. (43) 4th Trimestre, 1899.

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* Illustrated.

NEW BOOKS OF THE MONTH.

Unless otherwise specified, books in this list have been donated to the Library by the Publisher.

RAILROAD CONSTRUCTION.

Theory and Practice: A Text-Book for the Use of Students in Colleges and Technical Schools. By Walter Loring Webb, Assoc. M. Am. Soc. C. E. Cloth, 9 x 6 ins., 456 pp., illus. New York, John Wiley & Sons, 1900. $5.00.

This book is written primarily for students of railroad engineering in technical institutions. The contents are: Railroad Surveys; Alignment; Earthwork; Trestles; Tunnels; Culverts and Minor Bridges; Ballast; Ties and Other Forms of Rail Support; Rails; Rail-Fastenings; Switches and Crossings; Appendix. The Adjustment of Instruments.

THE STRENGTH OF MATERIALS.

A Text-Book for Manual Training Schools. By Mansfield Merriman. Second Edition, Revised. Cloth, 7½ x 5 ins., 124 pp. New York, John Wiley & Sons, 1898. $1.00.

In this treatise an attempt has been made to give a presentation of the subject of the strength of materials, beams, columns and shafts, which may be understood by those not acquainted with the calculus. The book deals mainly with strength, the subject of elastic deformations occupying a subordinate place. There is a chapter on the manufacture and general properties of materials, and also one on resilience and impact.

THE TECHNIC OF MECHANICAL DRAFTING.

A Practical Guide to Neat, Correct and Legible Drawing. By Charles W. Reinhardt. Cloth, 8 x 11 ins., 36 pp., illus. New York. The Engineering News Publishing Company, 1900. $1.00.

In the preface the author states that the book has been prepared for the purpose of furnishing to draftsmen a thoroughly practical guide to good mechanical drafting; no attention has been paid to the mathematics involved. Many of the illustrations used in this work have appeared in the columns of *Engineering News*.

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ADDITIONS.

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Eng., Mo. Valley Bridge and Iron Works,	Assoc. M.	Feb. 3, 1892
Leavenworth, Kans	M.	April 4, 1900
CREUZBAUR, ROBERT WALTER,	Jun.	April 2, 1890
Prin. Asst. Eng., Dept. of Finance, 280 Broad-	Assoc. M.	April 4, 1894
way, Room 55, New York City	M.	April 4, 1900
DART, JUSTUS VINTON,		
Charge of Highway Dept., Providence, R. I		April 4, 1900
DAVIS, ARTHUR POWELL,		
Hydrographer, U. S. Geological Survey,	Assoc. M.	June 7, 1893
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KELSEY, CLIFFORD STEPHEN,	Jun.	April 3, 1889
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215 West 125th St., New York City		April 4, 1900
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Conn	M.	May 2, 1900

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TOMPKINS, EDWARD DE VOE......Asst. Eng., Dept. of Bridges, New York City. Res., 372 Park Place, Brooklyn, N. Y.

DEATHS.

DAVIS, FRANK PAUL..............Elected Member Feb. 1st, 1888; died May 3d, 1900.

DE COURCY, BOLTON WALLER.....Elected Member Nov. 6th, 1889: died April 1st, 1900.

Vol. XXVI. MAY, 1900. No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

A PROPOSED METHOD FOR THE PRESERVATION OF TIMBER.

By F. A. KUMMER, Jun. Am. Soc. C. E.

TO BE PRESENTED JUNE 6TH, 1900.

In presenting these notes upon a proposed method for the treatment of timber to preserve it from decay, the writer fully realizes that they may seem to add but little to the mass of information already collected upon that important subject; but the experiments which are herein described and the direction in which they lead, indicate a line of improvement in the preservation of timber which will prove of interest.

So many figures have already been presented, proving the great need of a successful method of timber preservation and the immense value of such a process—could one be secured, giving practical results at a reasonable cost—that the means by which this result may be obtained may be at once considered without stopping to prove its necessity.

The experiments herein described, and the lines upon which the proposed process has been developed, were made, particularly with a

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

view of improving what is at present known as the creosoting process, it being believed that this process would command a much more extensive use if certain mechanical difficulties, now met with in connection with its application to timber, and particularly to railroad ties, could be removed. The improvement of this process was also attempted because of its acknowledged merit when properly applied. As at present applied, it possesses certain features which preclude its extensive use in the treatment of railroad ties. If, by a simplification or improvement of the process as now used, the objections hitherto raised to it may be overcome without sacrificing any of the advantages which it possesses, and if, further, this can be accomplished at no greater cost, the resulting process would commend itself to engineers generally.

The often-quoted report* of the Committee of this Society, presented in 1885, said in part:

"If the exposure is to be that of a railroad tie, creosoting is doubtless the most perfect process to use; but in view of the expense, it may be preferable to use a cheaper process, dependent somewhat upon the location."

It is probable that at the present time a greater expense for treatment would be warranted by the cost of timber, as well as by the general conditions of roadbed maintenance, particularly upon eastern roads. Without here entering upon a discussion of the expense for preservation which would be justified under given conditions, it would perhaps be better to discuss first the proposed process from a technical standpoint and, after arriving at its cost, determine to what extent a process of a given cost per tie could be used.

The creosoting process as at present applied in one of the largest, if not the largest, creosoting works in the world, is described in part as follows in the printed publications of that company:

"The Bethel process, the first to use dead oil of coal-tar, consisted of drying the timber naturally, then applying cold vacuum. This takes too long a time, as large timbers, to dry in this manner, take from 3 to 6 months. Consumers of creosoted timber in this country cannot wait for timber to dry naturally, and so steam heat was introduced * * * to hasten the process. By the use of steam, assisted by superheating, the process is reduced to a small fraction of time. By our process timber can be used fresh from the stump or water, and in a few hours the sap and moisture extracted by means of our

* *Transactions*, Am. Soc. C. E., Vol. xiv, page 290.

immense steam boilers and coils, also by our powerful vacuum pumps, and be prepared for impregnating with oil. * * *

"The timber is first loaded on cars and run into the cylinders which are then hermetically sealed with immense iron heads. Steam is then admitted into the cylinder and surrounding the timber, superheated steam is also introduced into the cylinders by means of large coils so that it does not come in contact with the timber, and the heat is maintained until the timber is heated all through at a low temperature so as not to injure the woody fibers. The cylinder is then freed of all vapors, and the vacuum pumps are put to work to exhaust all the sap and moisture, which is then in the shape of vapor, from the cylinder. Heat is maintained in the coils to prevent the vapor from condensing and thereby remaining in the timber. As the vacuum pumps are constantly removing the hot vapor from the timber it is absolutely necessary to keep the heat above the condensing point. To do this requires practical experience and means of knowing what such heat is, and as said before, these two parts of the process are the most important, and, if properly done, the oil will be readily forced into the timber. After this has been done the oil is admitted into the cylinders while they are under vacuum, and when all the air has been withdrawn they are subjected to pressure until the requisite amount (which is determined by correct gauges and thermometers) has been forced into the timber, which, if the timber has been properly prepared, is only a small part of the process; but if this has not been well done, the oil cannot be put into the timber. The cells of healthy timber are full of different substances, which, when subjected to heat, can be changed into vapor, and unless the vapor has been completely removed you cannot force the oil into the timber, no matter how long the pressure is applied. It is only by practical knowledge and delicate instruments that we determine when the heat has reached the center of the timber, and the vapor there formed has been removed.

"There will be no decay in any part of the timber that has been permeated with the oil, but to have all parts saturated is expensive and useless; for, after the timber has been thoroughly treated by the heat and vacuum process it will last a long time without any oil, and if the crevices and pores are sealed up with the oil to a sufficient depth the timber is as good as if the whole part had been thoroughly permeated with the oil. The quantity of oil to be used should be determined by the use to which the timber is to be subjected."

Before making any comments upon the process just described, the writer will give the reasons for the decay of timber, and state exactly what steps should be taken to prevent this decay. Having done this, he will analyze the dead oil of coal-tar process as at present used, and see whether these necessary steps are carried out properly.

There are two causes for the decay of timber. *First,* through the fermentation of its contained sap, such fermentation being caused by the germs contained or developed in the sap. *Second,* through the introduction of germs of decay from without, through the action of air or water. As Mr. Curtis says:*

"Ties perish by mechanical destruction of the fibers under the rail; splitting by seasoning and cutting out of fiber by respiking; and by natural decay. In its normal condition, to resist these three influences, a timber is required which shall have sufficient hardness to resist the mechanical injury, a high degree of coherence between fibers, and a sap of such a chemical composition as will offer the least encouragement to the development of the fungoid life which destroys the fiber."

To treat a tie satisfactorily, it must first be sterilized throughout; removing at the same time as much of the contained moisture as is possible without injury to the fiber of the wood. Fermentation cannot then take place from within. It must then be treated in such a manner that germs of decay cannot enter from without, or, if they do enter, that the conditions for their development are highly unfavorable. It must further be provided that the antiseptic injected will not be dissipated through any agency, and, if the timber is to be subjected to the crushing effect of heavy railroad traffic, it must in some way be provided that the timber so protected will be able to resist the weights which will pass over it, without excessive crushing.

If a tie be subjected throughout to a sterilizing temperature of 212°, the agencies of fermentation or decay existing in the sap will be killed, and, if prevented from again entering the timber from without, decay cannot take place. This leads to the first important conclusion; namely, that complete sterilization throughout is a necessity in the treatment of timber. Without such sterilization no form of treatment can be satisfactory unless the timber be completely impregnated throughout its entire mass with an antiseptic material, which is rarely if ever accomplished with large timber. Such sterilization means fully 212° at the heart of the stick. Wood is a poor conductor of heat, and, to secure 212° or more at the heart, two ways are open: *First,* long exposure to a temperature of say 215 to 225° Fahr., which is expensive because of the time consumed; *second,* exposure to a temperature of 290 to 300° Fahr., which, under ordinary conditions, will

* *Transactions*, Am. Soc. C. E., Vol. xlii, p. 288.

injure the fiber of the wood, or at least induce extensive splitting or cracking. The necessity for sterilizing is not brought out in the description of the creosoting process previously given, and it seems to be a point, the vital nature of which is not generally recognized. Experiments made upon yellow pine ties, 6 ins. x 8 ins. x 8½ ft., having self-registering thermometers imbedded at the heart by means of an auger hole, 24 ins. deep, bored from the end, showed that with a temperature of 230° Fahr. in the cylinder, and at the surface of the tie, 178° Fahr. was secured at the center in two hours, while with 290° Fahr. at the outside, 249° Fahr. was secured in two hours. Starting with sterilization as an absolute necessity, it is evident that, if such a temperature as 290° can be used without injury to the timber, complete sterilization will be effected, and in a reasonably short time. It would be well to state here that, with a few exceptions, the experiments were made on long-leaf yellow pine of the best merchantable grade, the exceptions being several experiments upon white oak timber, which showed that it is not capable of treatment by such a process, owing to its great tendency to split and crack under high temperatures.

The method adopted to render the use of such high temperatures possible in the treatment of yellow pine is the application of pressure simultaneously with the application of heat. When the doors are closed the heat is raised to 215° Fahr. without pressure, taking one hour to accomplish this, and kept for one hour at 215° without pressure. This is for the purpose of getting rid of the moisture. Then the heat is increased, pressure is applied, and both are raised gradually, to avoid injury to the fiber, for two hours, until the heat has reached about 285° and the pressure about 90 lbs., and both are held there for one hour. The heat is then shut off, and the tanks are allowed to cool gradually for one hour. At the end of this time the heat is reduced to 250° and the pressure to about 40 lbs. The pressure is then blown off and the heat still further reduced. Vacuum is then applied until about 26 ins. is reached, and while under vacuum the mixture is run into the cylinder at a temperature of 175 to 200° and hydraulic pressure applied reaching 200 lbs. per square inch, and kept at this point until the desired amount of the mixture is absorbed. The liquid is then run off, and the wood is placed in another cylinder, and milk of lime at a temperature of about 150°, is run in, and hydraulic

pressure of about 200 lbs. is applied for from one-half to one hour. The following is a condensed table of the treatment:

1 hour to reach	215°	Fahr.
1 hour under	215	"
2 hours (heat and pressure applied gradually) to reach	285	"
1 hour (90 lbs. pressure) at	285	"
1 hour cooling.		
1 hour vacuum (26 ins.).		
½ hour filling cylinders with mixture.		
3 hours pressure (average 8 to 10 lbs. absorption).		
½ hour emptying tanks.		
1 hour under lime and pressure treatment.		

The time will be increased or decreased, in keeping with the amount of absorption required.

The treatment of "green" lumber herein mentioned consists of simply putting the lumber in the cylinders and raising the heat to about the temperature of the liquid—say 200°—and then running the liquid in and keeping it there from 3 to 4 hours. Of course, this does not sterilize it or give as good absorption.

After the preliminary treatment, the temperature of the interior of the cylinder is raised to 285 or 290° Fahr. If this temperature were maintained at atmospheric pressure, not only would the sudden and rapid evaporation of the remaining moisture in the stick induce splitting and cracking, but the volatile oils in the wood would be driven off, and this is not desirable, inasmuch as they play a not unimportant part in the preservation of the timber. Air pressure is therefore maintained in the closed cylinder at about 80 to 90 lbs. per square inch, so that while the tie is sterilized by heat, the natural oils are not driven off, and splitting and cracking does not result, while the heat is at the maximum.

One of the benefits of the vacuum is to exhaust the air which has been forced in while the wood was under pressure, and which, if not removed, would have to find its way out subsequently either through the mixture or by creating cracks.

Mr. J. W. Putnam, in a letter to Octave Chanute, M. Am. Soc. C. E.,

Chairman of the Committee on the Preservation of Timber, in 1885, wrote as follows:*

"The most carefully conducted experiments indicate that there is no decay without fermentation, and no fermentation without germs. If a piece of timber be cut green and thoroughly coated with paint, it will soon be destroyed by what is called dry rot. If a similar piece be heated through to 225° Fahr., and a sufficient amount of oil be forced in to form an impervious coating, no decay will take place until that coating is broken."

This not only evidences the necessity of complete and thorough sterilization, but brings us to the next step in the process, the impregnation of the timber with the antiseptic material. Having thoroughly sterilized the tie, the natural continuance of the process is, of course, the application of the vacuum, to remove from the pores of the timber the air or vapor which they contain and admit the preservative fluid under this vacuum.

The preservative fluid used is not creosote, or dead oil of tar alone, but consists of dead oil of coal-tar, 38%; formaldehyde, 2%; and melted resin, 60%, by weight. The specific gravity of the resulting mixture at 300° Fahr. is 1.068. The resin is used to render the mixture absolutely waterproof, the formaldehyde to strengthen the antiseptic nature of the compound, necessarily somewhat reduced by the reduction in percentage of the dead oil itself. Upon this point the following extracts† from a letter by Mr. Edward R. Andrews to Mr. Chanute will be of interest:

"Creosote oil is a distillate of coal tar—a residual product in the manufacture of coal gas. Chemists have procured from coal tar a vast number of sub-products and combinations of great usefulness in dyeing, etc. The three principal coarse products of coal tar are the light oils, the heavy oils and pitch, all the results of distillation.

"The light oils (lighter than water) evolve in the distillation at a temperature of 360 to 480° Fahr. From these all the aniline colors are made. They are expensive and have no value whatever in wood preservation. The heavy oils (heavier than water) are distilled at a temperature of from 480 to 760° Fahr. These are the so-called creosote oils, and contain all the constituents of the coal tar useful in wood preservation. After the creosote comes the pitch. Creosote contains about 5% of tar acids, *i. e.*, carbolic, cresylic and other acids, but the bulk is made up of semi-solid oils and naphthaline.

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* *Transactions*, Am. Soc. C. E., Vol. xiv, 1885, p. 337.

† *Transactions*, Am. Soc. C. E., Vol. xiv, Appendix No. 14, p. 341.

"Wood preservation by the metallic salt processes is solely chemical. Earlier, it was claimed that the zinc chloride, etc., formed insoluble chemical combinations with the albumen contained in the sap wood. Now, it is generally allowed that no such combinations are formed, but that the value of metallic salts as antiseptics depends upon their continual presence in the woods, and as they are readily dissolved out of the wood their effect is only temporary. The life of wood is prolonged by their use, when skillfully applied, yet in moist places they quickly lose their efficacy.

"The creosoting process is both chemical and mechanical. Besides the carbolic and other acids, it contains many other well-recognized antiseptic constituents; but it is probable that the very long life of timber secured by thorough creosoting is due far more to the fact that the pores of the wood are filled up with the thick, gummy. insoluble oils and naphthaline, and thus keep out air and water, which contain the germs of decay. That such is the case was conclusively shown by M. Roltier, a Belgian chemist, and later, in 1866, by M. Charles Coisne, Chief of Section of the State Railways of Belgium, and Superintendent of the Creosoting Works.

"By the latter, two series of experiments were tried, during a period of five or six years, in burying in a compost heap made of decaying wood, manure, etc., shavings impregnated with creosotes containing different percentages of carbolic acid. The results showed that shavings saturated with carbolic acid alone were entirely decayed, and those saturated with the distillates at the highest temperatures which contained no carbolic acid whatever were perfectly sound.

"Experience with the metallic salts and the results of above experiments indicate that to preserve timber something more is required than an antiseptic for the purpose of coagulating the albumen. The very small percentage of albumen contained in the sap wood probably ferments readily and may originate decay; but the agencies of fermentation introduced into exposed timber by the air and water absorbed by the wood are vastly more dangerous than the seeds of decay contained originally in the wood itself.

"During the past hundred years almost every imaginable substance has been proposed as a preservative of wood, yet it may be that inventors are still at work, if so, their attention would be best directed to such methods or materials as would close the pores of wood to air and water."

This would indicate that creosote is more valuable because of its function in filling the pores of the wood and excluding air or moisture containing germs of decay than because of its direct antiseptic action. To a considerable extent, this is doubtless true, but the writer thinks that the ideal preservative should be both antiseptic and waterproof,

and should possess the latter characteristic in the highest obtainable degree. To secure such a material was the chief object of the experiments which resulted in the process herein described, and the compound finally fixed upon consists of approximately 38% by weight of dead oil of coal-tar, 60% of melted resin and 2% of formaldehyde. The idea of introducing the resin was to obtain a substance in the highest degree waterproof, and there are few substances known more highly waterproof than pine resin.

This mixture is perfectly fluid at 300° Fahr., and, by some peculiar action not readily explainable, seems to be more readily forced into the wood than the creosote oil alone. There are a number of very important advantages gained by the use of the resin in connection with the dead oil. Timber treated by the ordinary creosoting process is more or less spongy and soft, and while the dead oil is not generally regarded as soluble, it is still subject to dissipation by the action of water, as is well known in the case of treated piles subject to the washing action of salt water. Experiments made by immersing in water timber untreated and treated by the process just described, showed results as follows:

One piece creo-resin timber, 6 ins. x 8 ins. x 8 ft., immersed in water 27 days, absorption 1¾ lbs. One piece, same stick as above, untreated, absorption 11¼ lbs. The increase in weight of the first stick was largely due to water clinging to the surface.

Timber treated with dead oil in conjunction with resin is practically absolutely waterproof and very much harder than the untreated timber, instead of being softer and more spongy, as is the case with creosoting proper. The introduction of bacteria from without should be prevented by a process as highly waterproof as it is possible to make it, and this process should use a material which will not only have these waterproof qualities, but will be so antiseptic in nature that the possibility of the existence or development of decay germs, if such germs by any means gain an entrance into the timber, should be reduced to a minimum. In fact, the conditions should be such as to absolutely preclude any possibility of their life or development. To secure these results, the combination of resin, creosote oil and formaldehyde is especially well adapted. The material weighs about 8.9 lbs. per gallon.

Experiments were made upon pine ties cut in two, one half vulcanized, the other half not vulcanized. The results are given in Table No. 1, and show the difference in absorption.

TABLE No. 1.

No.		Maximum Heat.	Pressure in Lbs.	Pounds per cubic foot.
13 C.	Green.	285°	90	8.38
13 C.	Vulcanized.	285°	90	6.5
16	Green.	275°	80	7.14
16	Vulcanized.	275°	80	10.66
17	Green.	285°	85	8.48
17	Vulcanized.	285°	85	6.18
18	Green.	270°	80	6.75
18	Vulcanized.	270°	80	9.45
22	Green.	275°	90	5.23
22	Vulcanized.	275°	90	7.9

These specimens were treated with various degrees of heat and pressure, with and without vacuum, and in all cases they showed an increased absorption in the vulcanized over the unvulcanized timber.

Other experiments were made upon ties, 6 ins. x 8 ins. x 8 ft., of long-leaf yellow pine, cut into three pieces, two of which were treated and the third left untreated. The results are shown in Table No. 2, the treated samples, 1 and 1-A, 2 and 2-A, etc., being the two pieces from the same stick.

TABLE No. 2.

	ABSORPTION PER CUBIC FOOT, IN POUNDS.				ABSORPTION PER CUBIC FOOT, IN POUNDS.		
	Above 10 lbs.	Bet. 8 and 10 lbs.	Below 8 lbs.		Above 10 lbs.	Bet. 8 and 10 lbs.	Below 8 lbs.
1	10.68			14	18.5		
1-A	10.12			14-A	14.62		
2	10.12			15	15.18		
2-A	10.28			15-A	15.74		
3		9		16	16.81		
3-A		9		16-A	18.5		
4			7.87	17		9.56	
4-A			7.87	17-A		9	
5			7.3	18		8.43	
5-A			7.3	18-A		8.43	
6			5.62	19	15.18		
6-A			5.62	19-A	15.18		
7	11.18			20			7.87
7-A	18.5			20-A		9	
8	12.96			22	18.5		
8-A	14.62			22-A	14.62		
9			7.87	23	16.81		
9-A			6.75	23-A	16.87		
10			6.75	24	12.93		
10-A			6.75	24-A	11.81		
11		9.56		25		9	
11-A			7.87	25-A		9.56	
12	12.93			26	18.5		
12-A	18.5			26-A	16.87		
13	12.37						
13-A	11.81						

As regards the spongy condition of the wood, after treatment by the creosoting process, an inspection of timber treated by the proposed process readily demonstrates that the timber is not in any way softened. The surface is subjected to treatment by the application of milk of lime after the creosote-resin has been injected. This tends to solidify the mixture of resin and creosote oil. Solidification to a point of brittleness is not effected, but solidification is effected to such a degree that the resulting surface of the wood is materially harder than that of untreated timber, instead of being softer, as is the case with creosoted timber. The writer has made tentative experiments to prove this hardening effect by striking treated and untreated sections of the same tie with a hammer. The greater density and hardness of the surface of the creosote-resin tie (about 50% increase in density near the surface) being readily apparent from the way in which it resists crushing under the hammer, the force of the blow causing the hammer to rebound from its surface instead of sinking in and crushing the fiber, as was the case with the untreated portion of the same tie.

Obviously, if it were commercially practicable to impregnate a railroad tie, of, say, long-leaf yellow pine, throughout its mass with creosote oil, no decay would take place in the timber so long as the oil remained in it. But such a treatment is impracticable, both because of its cost, and of the mechanical difficulties attending it. We therefore are obliged to fall back upon the plan outlined by Mr. Putnam in the letter previously quoted, namely, to secure complete sterilization, and to then inject sufficient of the preservative to effectually protect the remainder of the stick. This may be done by the injection of 8 to 10 lbs. per cubic foot in a standard-sized tie, 8 lbs., giving a penetration averaging at the center of the tie about ¾ to ⅞ in., and 10 lbs., ⅞ in. to 1¼ ins. at the center, with from 6 to 8 ins. inward from the ends of the stick in each case.

A tie treated by the above process with, say, 10 lbs. of preservative per cubic foot possesses these advantages: It is thoroughly sterilized—a result not generally secured by the creosoting process; the outer fibers of the wood are filled with a material more highly antiseptic than dead oil of coal-tar, practically water-proof, and, when hard, considerably better able to resist crushing or cutting than is either the untreated or the creosoted timber. In fact, the spongy nature of creosoted timber, especially as the timber usually chosen for creosot-

ing is naturally soft, has been one reason why creosoted timber has not been more extensively used under heavy traffic. The experience of the Lehigh Valley Railroad was along this line. A tie treated by the process in question also possesses much higher spike-holding qualities—considerably higher than the untreated stick.

The writer has confined these remarks particularly to the subject of railroad ties; but it should not be overlooked that a large part of the business done by the creosoting works at present is upon piles and dock timbers; and the failure of such timbers when attacked by the teredo is invariable due to the washing out of the creosote oil by the action of the salt water. It would therefore be readily practicable to creosote piles, giving them either at the same time, or subsequently, a resin treatment to absolutely retain the creosote in the wood, and therefore the chief objection to creosoted piling would be at once removed.

The question of wooden paving blocks is also one having, in the writer's opinion, great possibilities. Wooden paving blocks, creosoted, are in extensive use in London and continental cities, and have given very favorable results in this country wherever properly used. If we add to the preservative effect obtained by the creosote oil the additional and most important benefit of so hardening the fibers of the wood that its resistance to wear is materially increased, we have realized a most material gain in the treatment of such blocks. In fact, if a treated pine paving block, protected against decay by dead oil of coal-tar, impervious to moisture and having considerably greater density and resistance to abrasion than either the creosoted or untreated timber, can be laid at a cost not greater than that at present charged for asphalt, it is the writer's opinion that such a pavement will possess all the advantages which asphalt possesses, and will be at the same time both noiseless and not slippery; thus removing two of the gravest objections to asphalt pavement. The writer thinks that such a pavement would have a longer serviceable life than asphalt—at least such as is generally laid—and on streets of a certain class be exceedingly desirable.

To revert once more to the question of tie preservation: With creosoting, as well as with the proposed improved process, the amount of impregnation per cubic foot is regulated by the specifications under which the work is done. For railroad ties the general practice is about

10 lbs. Whether pure creosote or creosote and resin are used, this specification gives a depth of penetration at the center of the tie averaging nearly 1 in., and at the ends of the tie a complete penetration from the end of the tie of from 8 to 10 ins. Both with creosoting and with this process, the depth of penetration depends not only upon the amount of material injected per cubic foot, but also upon the nature of the timber and upon the individual qualities of different sticks of the same kind of timber. The writer has suggested as a possible disadvantage to such processes—whether creosoting or improved creosoting—the likelihood of the splitting of the tie taking place after treatment, thus allowing the introduction of germs into the heart of the stick. A careful examination and sawing of some fifteen to twenty ties, treated for periods extending up to 6 months, has shown no instance in which new cracks had opened in the timber and extended beyond the treated portion of the tie. Should this be the case after several years' service, the writer believes it to be true that the vulcanized heart of the tie treated by the improved process would better resist the action of decay than the untreated and unsterilized heart of the creosoted timber.

Further, it seems probable that water containing germs would to some extent get rid of such germs in passing through that portion of the timber which is antiseptic in its action. The results obtained with creosoted timber up to the present time do not indicate that such cracking or splitting, and consequent decay, are among the serious defects of the creosoting process. It seems reasonable to conclude that a tie which has been subjected to the continued high sterilizing temperature has developed splits in those weak portions where splits would otherwise have taken place under natural conditions, and these splits or cracks, developed in the earlier steps of the process, are filled with the antiseptic and waterproof material during the later stages, thereby excluding permanently both air and water.

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IRRIGATION STUDIES.

By ELWOOD MEAD, M. Am. Soc. C. E.

TO BE PRESENTED SEPTEMBER 5TH, 1900.

The Office of Experiment Stations of the United States Department of Agriculture is now engaged in a comprehensive study of the methods of distributing and using water in irrigation. The results of these studies for 1899 have been compiled, and will shortly be published by the Department. Some of the methods used in these investigations, and the results of the measurements to determine the losses from seepage and evaporation in canals and the duty of water utilized have been extracted from the report and are presented in this paper.

The investigations described deal with problems which sorely perplex the irrigators and canal builders of the arid West. Their comprehensive study is a new feature of national aid to irrigation development in this country. Heretofore, the leading object of such aid has been to promote the construction of new canals, to show how much land there was above existing ditches which could be reclaimed, and the benefits which would come from such reclamation. It is believed that this investigation will tend to secure these ends, but its primary

NOTE.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers with discussion in full will be published in *Transactions*.

purpose is to promote the welfare of the people living below the ditches already built, to render the farms now irrigated more profitable, to lessen the controversies over the distribution of water and secure its more systematic and economical use.

It is the opinion of those best informed that a better understanding of the existing situation must be had before we can wisely plan for future development. Controversies over the use of streams should be ended before an attempt is made to greatly augment such use. The claims to water, for existing and prospective ditches on many streams, amount in the aggregate to many times the supply. The character and extent of the rights now vested must determine what is to be done in the future. In the pioneer stages of western irrigation, the importance of these questions was lost sight of. The owners of the earlier ditches from a river did not need to consider how much water was used or how much was wasted, since neither had as yet made any serious reduction in the supply. Every transaction which had to do with the disposal of streams was marked by a lavish prodigality. Ditches diverted more water than was used; their owners claimed more than they could divert, while decrees gave appropriators titles to more water than ditches could carry and many times what the highest flood could supply. Little was known of the quantity of water needed to irrigate an acre of land, and in the absence of such information the ignorance and greed of the speculative appropriator had its opportunity.

The contracts which control the distribution of water from canals were framed by people to whom the whole subject of irrigation was strange and new. It often happens, therefore, that they do not promote the best interests of canal companies or meet the necessities of users. The laws which govern appropriations of water from streams have in most cases no relation to the actual practice of irrigators, and, therefore, fail to secure either the systematic distribution or best use of the available supply.

Before the period of crude structures and still cruder ideas had ended, it began to be manifest that the reclamation of arid lands involved more than the overcoming of physical obstacles. It has been found easier to dig ditches than to distribute the water they carry, and to plan headgates and flumes than to frame just laws for establishing titles to water or dividing rivers among rival claimants.

The many thousands of miles of canals and laterals in the irrigated regions of the United States have reclaimed an area approximately as great as the State of New York, every acre and almost every square foot of which has to be artificially moistened from one to ten times each year. During the growing season this requires the services of an army of men to protect and regulate headgates, patrol the banks of canals and adjust the measuring boxes of users. The success or failure of these canals is a matter of more than local interest. Much of the money expended in their construction came from the East. The small savings of thousands of thrifty New England people have been invested in stocks and bonds of irrigation companies, a single agency in Colorado having invested $15 000 000 in this class of securities for these customers. The failure of a canal company to find customers for the water carried, or to obtain water for the customers it has, affects many others besides the water user. His is the immediate loss, but sooner or later the holder of the stocks or bonds of the canal company finds that he, too, is suffering through delayed or defaulted interest payments. The justice and efficiency with which the stream is divided, and the economy with which the water is used, may augment or reduce the incomes of many thousands of eastern as well as western homes.

The Distribution of Water Among Users.

Traffic in water is carried on under many peculiar and perplexing conditions. No matter from what source the supply is received, whether it is stored in reservoirs, pumped from wells or taken from rivers, the distribution of water in irrigation is subject to unending uncertainties. Streams rise and fall with every passing cloud. The torrent of to-day may be a dry channel a month hence. Wells which cannot be exhausted in April are often empty in June. Even after water has passed the headgate and is safe from outside interference, the waste and loss continue. It disappears through the bottom of the canal by seepage, and into the air by evaporation. The same vicissitude attends its use. As much water may escape from the lower side of the field of a careless irrigator as sinks into the soil. The waste from badly built laterals or poorly prepared fields does much to limit the acres which a canal can serve, and hence the income it can be made to yield.

This commerce in water has been created by men born and reared in regions of ample rainfall, and without prior training or experience in dealing with the problems of irrigation. They had to learn by trial how to frame satisfactory contracts for the disposal of water from canals, and how to use that water properly when delivered. From the construction of the first small furrows in Utah and California up to the present, the growth in acres irrigated has been accompanied by an equally important evolution in methods. The fixing of a unit of measure to be used in delivering water to users will serve to illustrate this. It could not be sold by the pound or by the ton, nor were there any devices at hand for its measurement or delivery by the gallon. Farmers were at a loss to know how much to buy, and canal companies as ignorant of how much they could sell or how to measure it when sold.

The Units of Volume Used in Measuring Water.

The "Inch."—In a number of the arid states placer mining was an important industry before irrigation began. Miners, in measuring water, used the "inch." This is the volume which will flow through an inch-square orifice under a uniform and designated pressure. Later, the pressure to be used, and the manner in which the size of the orifice was to be increased or diminished, was, in a number of states, fixed by law.

"Water sold by the inch by any individual or corporation shall be measured as follows, to wit: Every inch shall be considered equal to an inch-square orifice under a 5-in. pressure, and a 5-in. pressure shall be from the top of the orifice of the box put into the banks of the ditch to the surface of the water; said boxes or any slot or aperture through which such water shall be measured shall in all cases be 6 ins., perpendicular, inside measurement, except boxes delivering less than 12 ins., which may be square, with or without slides; all slides for the same shall move horizontally, and not otherwise; and said box put into the banks of ditch shall have a descending grade from the water in ditch of not less than one-eighth of an inch to the foot."*

In those sections where irrigation succeeded this form of mining, irrigators generally adopted this unit. In many respects it is entirely satisfactory. Where the flow is controlled by a device of reasonable accuracy it is a convenient method of delivery for canal companies, and satisfactory to users, because they can tell at a glance whether or

* General Statutes of Colorado, Sec. 3472.

not the quantity contracted for is being delivered. It is not suited, however, to the measurement of rivers or to the regulation of their division among large canals, as the prescribed conditions cannot be produced on a large stream of water. There are canals which carry 125 000 ins. To measure this volume, under the conditions prescribed in the Colorado statute, would be practically impossible. The use of the term "inch" has been unfortunate. Many farmers have confused this unit with the square inch or the cubic inch, and it frequently happens that the inches of water bought or sold are determined by measuring the cross-section of a ditch or lateral, paying no attention whatever to either grade or velocity. In one case a state law confuses cubic inches with the continuous flow from an inch-square orifice.

The Cubic Foot per Second.—When it became necessary to gauge streams and to measure the volume of large canals, it was manifest that some other unit of measurement that the miner's "inch" had to be used. The cubic foot per second is the unit which has passed into general use. This unit has the double advantage of showing precisely what is meant, and being well adapted to the measurement of large as well as small volumes of flowing water. It is the most satisfactory unit which can be used in dividing rivers or in measuring the flow of large canals where the flow is continuous. There is, however, an objection to its universal use in water-right contracts, or in decrees establishing rights to water. Where decrees or contracts provide for the measurement of the quantity received as a continuous flow, it presupposes that irrigators use water in this manner. This is not in accord with the best practice. Irrigators do not need water all the time. Few use it half the time. If they are required to pay for a continuous flow, they usually pay for something they do not get, and always for what they do not need. If they are allowed, as an equivalent of a continuous flow, to take a larger volume for a shorter time, a different unit of measurement is desirable; because it is not a stream of a particular size, but the total volume received, which is paid for.

The Acre-Foot.—The growing recognition of the fact that a continuous flow of water does not correspond to the needs of irrigators has recently brought into use another unit of volume, the acre-foot. It contains 43 560 cu. ft., or enough to cover an acre 1 ft. deep. It is a convenient unit for selling stored water, because it can be used to measure the capacity of reservoi[illegible]

Contracts in which the acre-foot is used provide for the delivery of water on the demand of the irrigator, or at intervals rather than in continuous flow; and canal companies have hesitated about adopting this unit, because of a fear that satisfactory arrangements for delivery could not be made, and that more water would be called for at some times than the canal could supply, while at other times the entire volume would run to waste.

Wherever the acre-foot has been adopted it has proven acceptable to irrigators, because they share in the benefit resulting from care and skill in distribution.

Form of Water Contracts and Benefits of Rotation.

Contracts for supplying water take many forms. In some cases they are deeds to the water delivered; in others they purport to transfer a perpetual right to a specified quantity or to a stream of a specified size; in others they agree to provide water for the irrigation of a specified number of acres for one year; while a few provide for payment for the quantity actually used, fixing a maximum quantity which may be demanded in one season. At first these contracts were largely governed by the ideas of the managers of the canals, but sufficient experience has now been had to make certain forms of contracts preferred in behalf of public as well as private interests.

Contracts which provide for the delivery of a uniform, constant flow are, as a rule, wasteful of water, and are not in the interests of either ditch companies or the public. Contracts which charge for the acres irrigated, without regard to the volume used on these acres, are a temptation to extravagance on the part of the irrigator. The canal company which adopts such contracts resembles the grocer who would agree to supply his customers with a year's provisions at so much per head, with no restrictions as to quantity or kind of goods which might be called for. On the other hand, contracts providing payment proportioned to the quantity delivered, and for delivery in amounts which can be most efficiently distributed, cannot fail to lead to economy in the use of water, and consequently to a high duty. Under such a system the irrigator is benefited by his saving, and pays for his waste. Such contracts can only be used in connection with a system of rotation in delivery to irrigators. This rotation benefits the canal company as well as the irrigator, because it lessens the loss

from evaporation and seepage. A canal 60 miles long could be divided into three sections of 20 miles each, and all the loss from seepage and evaporation on the lower 40 miles saved while the irrigators of the upper section were being supplied. In the same way, by keeping the full supply in the canal, water could be rushed through to users under the lower section with less loss than where the flow is depleted by laterals along the route. The greatest saving in rotation, however, would be made in the laterals. Where water is permitted to slowly dribble through continuously, the waste is enormous. By devising a system for grouping the laterals and inducing the irrigators therefrom to take water by turns, the engineer can do as much toward raising the duty obtained as the actual cultivator. The use of a unit which favors rotation between users leads also to rotation in the division of the river between the canals. The loss from seepage and evaporation is approximately the same whether the canals are full or only half full. When rivers are low, by running half the canals at a time with a full supply, nearly half the water ordinarily lost in transit is saved. As the loss from seepage and evaporation averages about 30% of the water flowing in canals, the water saved by such rotation is a material addition to the available supply.

Reasons for Investigation of the Duty of Water.

As the water required to irrigate 1 acre of land should be the basis for fixing the dimensions of works required to irrigate any number of acres, there is need to know approximately its amount. In order to plan for the just distribution of the volume entering the headgate, the losses in transit must be provided for. Until more is known than is now known about the time of year when the irrigation season begins and ends, the part of the discharge which must run to waste unless stored cannot be estimated. Until it is known how large an area an acre-foot of stored water will irrigate, and the returns which will come from such irrigation, the value of reservoirs will have no more substantial basis than individual judgment or conjecture, and no intelligent estimate can be made of the amount of money which can be profitably spent in their construction. Sooner or later, a knowledge of the duty of water becomes a necessity in any irrigated district. It is now urgently needed to settle disputes over water-right contracts, and to provide for their intelligent reconstruc-

tion. Thus far, it has been the uniform practice to make all rights to water perpetual and continuous. This is not the practice of European countries. Italy, France and Spain, each distinguishes clearly between rights to the summer and to the winter flow, the vernal and autumnal equinoxes being the dates when one begins and the other ends. The controversies which have recently arisen over rights to the winter flow of streams will doubtless soon lead to a similar distinction in western irrigation laws. A comparison of the duties secured under many of the canals where measurements were made last year leads to the belief that it will be possible through improved methods to double the average duty now obtained, so that the quantity now required for one acre will serve to irrigate two. If this can be accomplished it will relieve the scarcity under many canals, put an end to many controversies growing out of the existing scarcity, lessen the expense per acre for water, and increase immensely the productive and taxable resources of the arid States.

Believing that a more general understanding of the causes which increase or diminish the duty of water is one of the most urgent needs of irrigated agriculture, the determination of this duty was made a leading subject of the investigation.

Methods Used in the Investigation.

In carrying out this investigation laboratory methods will not answer; it must deal with the use of water on a large scale. The work requires the supervision of men of special training and wide practical experience. One of the chief difficulties encountered at the outset was to find the right men to take charge. Those engaged are, without exception, holding positions of responsibility and receiving ample compensation from other sources. The chief inducement for their taking part in this investigation has been the promotion of the public welfare. Through their interest and zeal a large amount of information was obtained which could not otherwise have been secured for ten times the actual outlay. It was left for the observers in each State to secure the co-operation of intelligent, practical farmers, and to arrange with them to measure the water used on their fields. In nearly every case this was easily accomplished. Every farmer connected with the investigation received the same instruction. It was, to use water whenever and wherever it was thought necessary, pro-

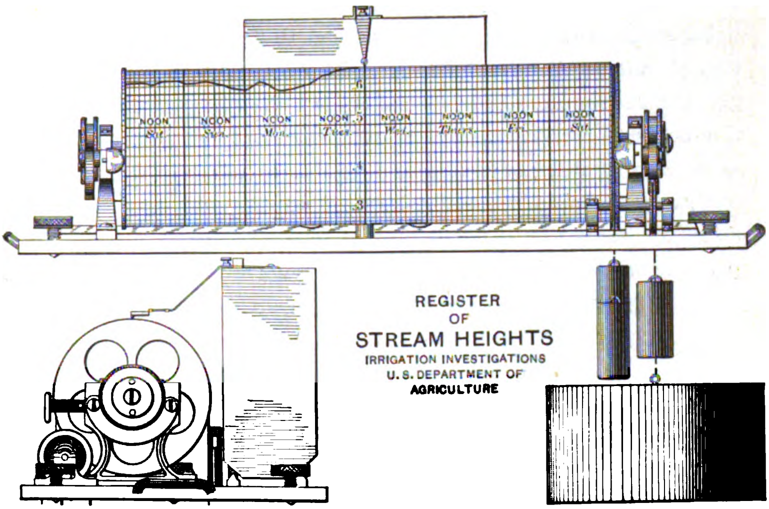
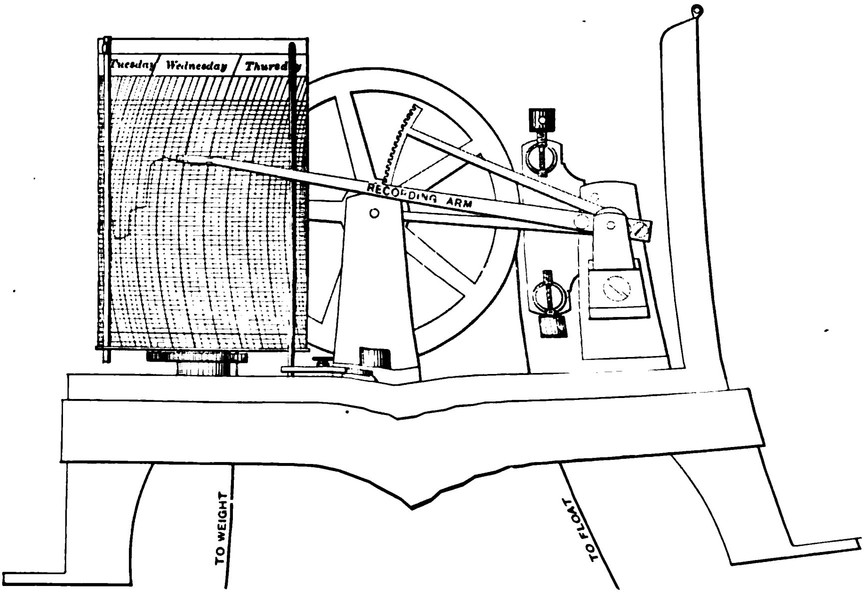


FIG. 1.



THE RICHARD FRÈRE REGISTER.
FIG. 2.

vided it could be had, and pay no attention to the fact that it was being measured. The results show that this was done. The description of canal systems and the methods which govern their operation, given in the reports of the special agents, show how direct is the relation between good management and a high duty of water. They also show how prolific of waste and loss is a badly drawn water-right contract. Records were also kept of rainfall and evaporation, and an effort was made in each case to secure as much information as possible on the following factors of the duty of water in irrigation:

The quantity of water required by different crops.

The length of the irrigation period in different sections of the arid region.

The agreement or divergence between the quantity of water used in irrigation in the different months of the growing season, and the rise and fall of streams during those months.

The benefits of reservoirs, and the percentage of the total discharge of streams which must be stored in order to utilize it all.

Losses in canals from seepage and evaporation.

Influence of different forms of water-right contracts in promoting economy or waste.

The returns from the use in irrigation of an acre-foot of water.

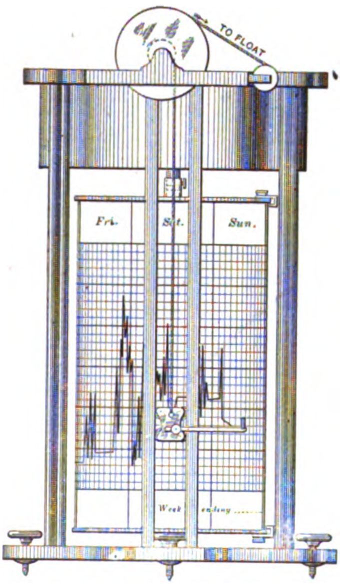
Instruments Used in Recording Amounts of Water Used.

In the study of the duty of water, and to record the quantity used, provision had to be made for an instrument which would be automatic and continuous in its operation. This was necessary because the quantity received by each irrigator from his lateral is subject to frequent change. Finding it impossible to use a meter to measure the volume delivered, it was decided to place a weir or flume in each canal or lateral, and then, by means of a suitable instrument, keep a continuous record of the depth of water delivered. Wherever possible, weirs were used, but in more than half the cases where they were used the results proved that flumes would have been more satisfactory.

A study of the registers in use showed that none was wholly satisfactory. The first requisite was a record on a natural scale, so that an inch rise or fall in the ditch would be so shown on the record sheets, thus enabling any farmer or ditch rider to determine at a glance

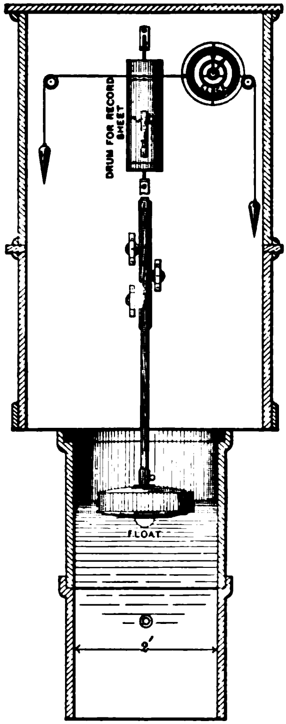
whether or not the instrument was working accurately, and, if not, to correct it without the computation required where the scale is reduced. Not being able to obtain registers of this pattern, one was designed. Its form is shown in Fig. 1.

In this instrument the rise and fall of the water in a ditch or lateral raises and lowers a float and counter-weight. This float and counter-weight are connected by a cord which passes over the end of a cylinder



THE WYOMING NILOMETER

FIG. 3.



REGISTER OF HEIGHT OF WATER
GAGE CANAL, RIVERSIDE, CALIFORNIA

FIG. 4.

which is revolved by the movement of the cord as the float rises and falls with the changes of depth in the stream. This cylinder carries a paper divided into rectangular spaces, the time divisions being parallel to its axis and the depth divisions at right angles thereto. The pen or pencil making the record is moved along this cylinder by clockwork, and passes from one end to the other in a week, when the paper is

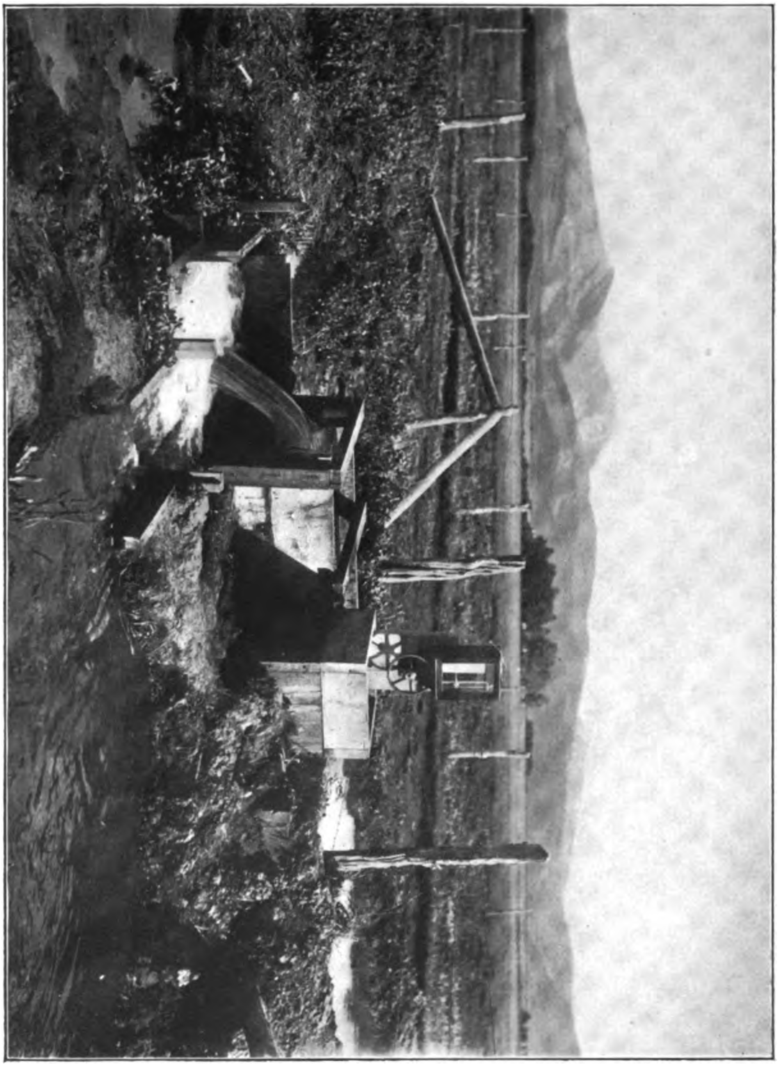
changed and the pen returned to the starting point. The only result of a wide variation in stream depth is an increase in the number of revolutions of the cylinder.

Owing to delay in the construction of these instruments, all observers could not be supplied with them, and a number of registers of other patterns were used. The Richard Frères register, Fig. 2, was utilized in the measurements in Arizona, the instruments being loaned by the University of Arizona. The Wyoming Nilometer, Fig. 3, was used at the Wyoming and Nebraska Stations. The Irving register, Fig. 4, was used on the Gage Canal in California. Plate XXIV show the method of placing registers used in weir measurements.

Relative Merits of Weirs and Flumes in the Measurement of Water.

Reference has already been made to the fact that some of the weirs did not prove satisfactory. This was due to the deposit of silt above them. Sediment investigations made during the season showed that certain southern streams carry, during floods, as high as 5% of solid matter in suspension, and that canals and laterals taking water from these streams have to be cleaned from two to three times each year. Even where the percentage was much less than this, the deposit of sediment was so rapid in some cases as to fill the lateral or ditch above the weir to a level with its crest in 24 hours. Where this happened, the velocity of approach became a disturbing factor, the influence of which could not be determined owing to the constant change of conditions. Some canal companies which use weirs, operate, in connection therewith, a sluicing device which removes the accumulated sediment once each day. The objection to this is that the conditions are never stable, and it is impossible to tell for what length of time the weir tables used agree with the actual discharge. The recent investigations in the flow of water over dams and over weirs, other than those with sharp edges, may aid in securing the adoption of a form of weir better suited to the sediment-laden waters of the Southwest than that used, but, so far as knife-edged weirs are concerned, there are few ditches in the Southwest where it is not possible to secure rating tables for flumes which will give much more reliable and accurate results. It was also found that, in the case of a number of canals, the grades were too small and the banks too low to secure the requisite fall

PLATE XXIV.
PAPERS, AM. SOC. C. E.
MAY, 1900.
MEAD ON IRRIGATION.



REGISTER FOR WEIR MEASUREMENTS.

below the weir, and in such cases flumes would not only be preferable but an inevitable substitute.

The most serious objection to the use of flumes is the labor of preparing an accurate rating table, and the fact that a current meter is required for doing this. The recent improvements in these instruments, by which both their convenience and accuracy have been increased, has made it a simple matter to prepare a discharge table for flumes in which the flow is reasonably uniform. With ordinary care this discharge can be determined within the limits of accuracy permitted by the meter used, and in the best instruments this error is as low as 1 per cent. This margin of error is below what is permissible in the delivery of water or attainable in this investigation. The relation of the volume discharged to the depth of water in the measuring

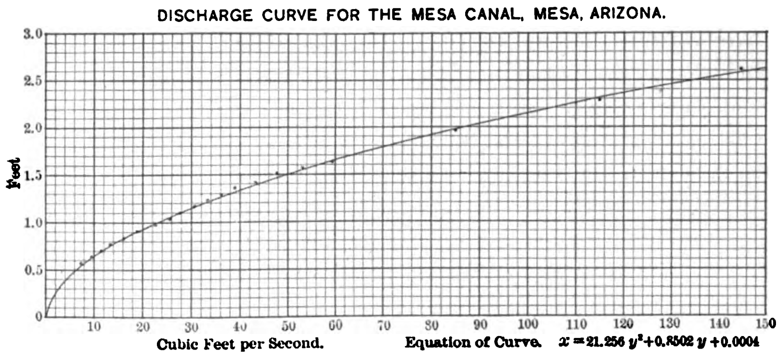


Fig. 5.

flume of the Mesa Canal, Arizona, is shown in Fig. 5. The dots show the gaugings. The curve is platted from the equation $x = 21.256\,y^2 + 0.8502\,y + 0.0004$, which is derived from the actual gaugings by the method of least squares.

The Unit of Measurement Used.

In the tables which follow, the acre-foot is the unit of volume used. This is usually expressed by giving its equivalent in the depth to which the water used would have covered the surface irrigated. This unit was chosen because it is definite, and because it affords a convenient base for the comparison of quantities used in localities where the period of use was not the same.

It is usual in discussions of the duty of water to take the cubic foot

per second as the unit of quantity, and the period during which a crop requires water to bring it to maturity as the time during which the flow of that volume continues. Thus, when it is said that the duty of water is 60 or 80 acres to the cubic foot per second, the statement implies that the quantity of water used during the season had been measured and the average volume used during this season amounted to 1 cu. ft. per second for each 60 or 80 acres irrigated. In order to make this expression definite, it is necessary that the duration of the irrigation period be known; but this varies so widely in different localities, and in actual practice in the same locality, that it is difficult to compare the results obtained. In a number of discussions of this subject the assumed season for the Rocky Mountain regions has been taken as varying from 100 to 150 days. The records kept last year show, however, that water was used from the Gage Canal at Riverside, Cal., throughout the entire year, while the canal at Wheatland, Wyo., was operated only 60 days, and water was used in irrigation a shorter time. Since the practice varies so widely, any attempt at fixing an average period would be wholly arbitrary. Even under the same canal the length of the season has to be assumed, because no two irrigators use water for the same length of time. The length of the irrigating period at the several stations is shown graphically in Fig. 6. Nor does the assumption of a continuous flow accord with practice, because on many canals a system of rotation in the delivery of water is already in operation, and even where the contracts provide for a constant delivery it seldom happens that irrigators use water in this way. When, therefore, in practice, 20 miner's or statutory inches are used on an acre for a day, and none at all for the next twenty days, it is an error to discuss the duty as though a single inch had been used all the time. In nearly all the northern States fully three times as much water is used in July as in August. Hence, a discussion which deals with the delivery of water as though the use was uniform during this period is likely to lead to serious mistakes in practice.

By the use of the acre-foot as the unit of quantity all the arbitrary assumptions involved in the use of either the "inch" or the cubic foot per second are avoided, and its use is equally correct and convenient whether the supply comes from streams, wells or reservoirs, whether it is continuous or intermittent, and whether it ends in two months or extends throughout the entire twelve. The flow of 1 cu. ft. per second

DIAGRAM SHOWING DURATION OF IRRIGATION PERIOD ON MAIN CANALS INCLUDED IN INVESTIGATIONS.

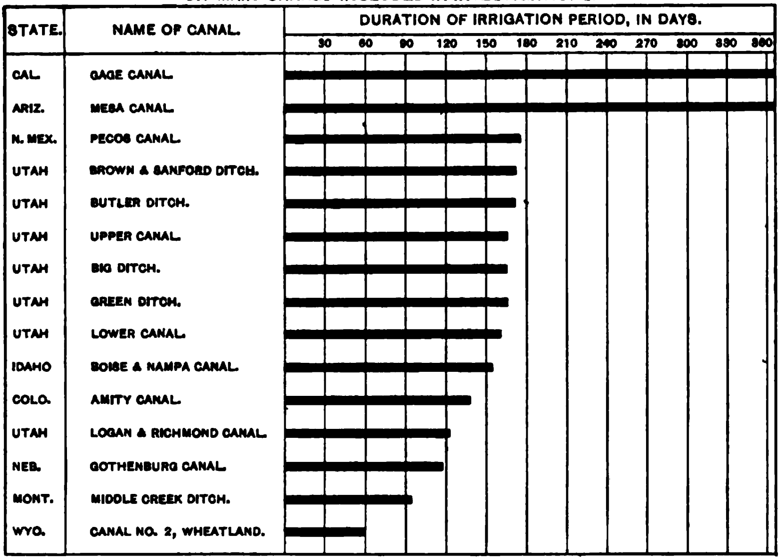


DIAGRAM SHOWING DURATION OF IRRIGATION ON FARMS WHERE WATER WAS MEASURED IN 1899.

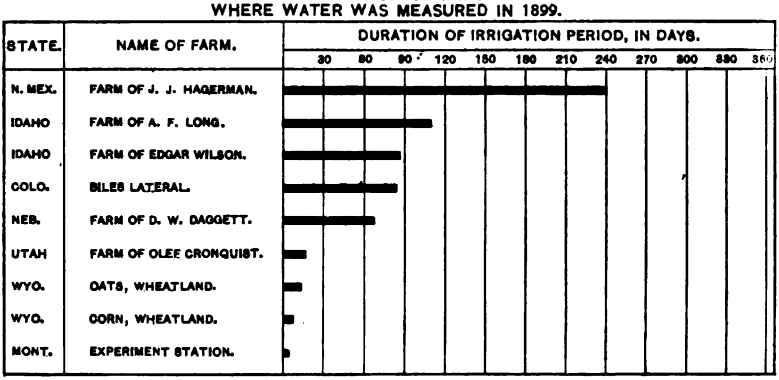


FIG. 6.

for 24 hours amounts to 1.98 acre-feet, so that the conversion of volumes from one unit to the other can be readily made.

SUMMARY OF MEASUREMENTS OF DUTY OF WATER.

The measurements made by observers show that the duties obtained vary from less than 1 acre-foot of water per acre irrigated to over 15 acre-feet on an acre; but these wide and seemingly eccentric variations in the quantities used were the results of manifest causes. Where water was distributed through well-built ditches and used by careful irrigators, there was a surprisingly close agreement in results even in widely separated localities. Table No. 1 will serve to illustrate this.

TABLE No. 1.—QUANTITY OF WATER USED, WHERE MEASUREMENTS WERE MADE ON SMALL CANALS OR SHORT LATERALS.

Location.	Depth of water used, in feet.
Cronquist Farm, Utah	2.60
Long Farm, Idaho	2.40
Gage Canal, Cal	2.24
Canal No. 2, Wyo	2.53
Vance Farm, Ariz	2.82
Biles Lateral, Colo	1.82
Middle Creek Ditch, Mont	2.10
Gothenburg Canal, Neb	2.57
Mean of all the above	2.31

An interesting comparison with the results in Table No. 1 is afforded by the mean of the duties on all the distributaries of the Ganges Canal for 1889-90, during the Khareef season, as reported in Buckley's "Irrigation Works in India." Here, the mean volume of water used in the irrigation of an acre of land was 121 970 cu. ft., equal to a depth of 2.8 ft. for the area irrigated.

Where the water was measured at the margin of the fields, there was a still higher duty than where measured at the heads of the laterals. Table No. 2 shows the duty obtained where all losses in distribution were eliminated, and nothing but the water actually spread over the fields was measured.

DIAGRAM SHOWING THE TIME OF IRRIGATION AND THE DEPTH OF WATER USED ON THE FARM OF OLEF CRONQUIST, LOGAN, UTAH.

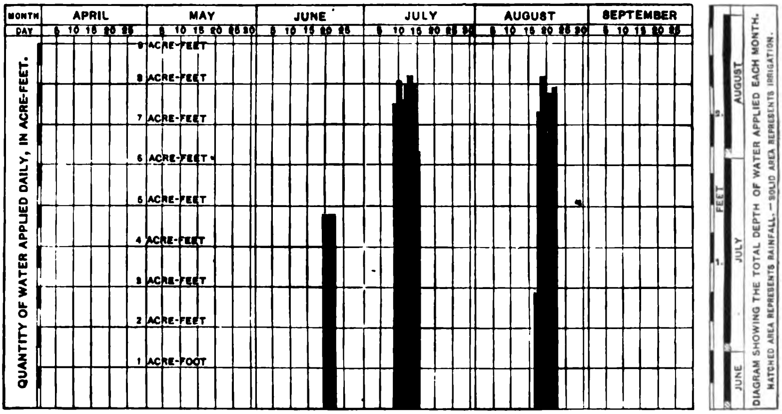


Fig. 7.

DIAGRAM SHOWING THE TIME OF IRRIGATION AND DEPTH OF WATER USED ON THE FARM OF A. F. LONG, NEAR NAMPA, IDAHO

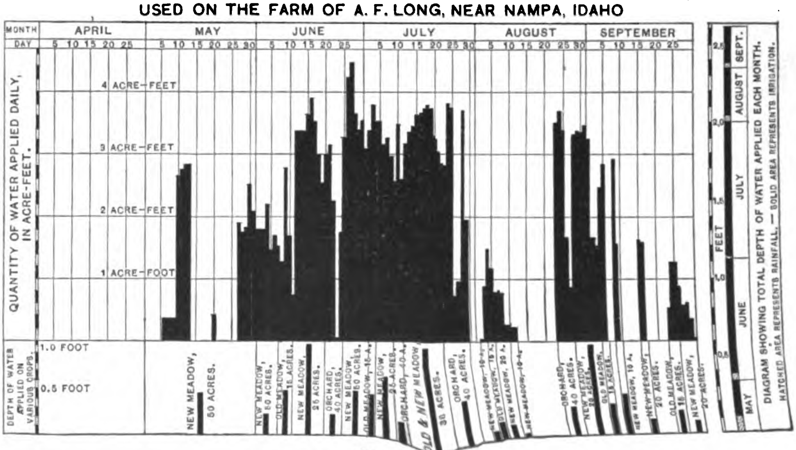


Fig. 8

TABLE No. 2.—QUANTITY OF WATER USED, WHERE MEASUREMENTS WERE MADE AT MARGIN OF FIELD WHERE USED.

Location.	Depth of water used, in feet.
J. Lateral, Wyo. (Oats)	1.55
J. Lateral, Wyo. (Corn)	0.70
Farm, Edgar Wilson, Idaho	1.48
Lowest Division, Gage Canal, Cal	1.78
Mean of measurements at Bozeman (Mont.), Experiment Station	1.20
Mean of all the above	1.34

The dates when water was applied, the length of the irrigation periods, the total acre-feet per day in use, the total depth of irrigation applied each month and the depth of rainfall for each month, are all shown graphically in Figs. 7, 8 and 9, for three of the stations named in Tables Nos. 1 and 2.

LOSSES BY SEEPAGE AND EVAPORATION.

The duties given in the foregoing tables were obtained on laterals, or on canals where the losses in transit were not large, and on fields where the water was measured at the margin. They therefore represent, approximately, the volume utilized. In practice, however, the losses in canals from percolation, leakage of flumes, evaporation, etc., are an important factor in fixing the average duty of water from a river or an extensive canal system. To determine this average duty, the volume should be measured at the headgate, and the number of acres it irrigates is the duty which canal managers have to consider in determining the area their works will irrigate. This duty is much lower than that obtained by measurements made on laterals or at the margins of the fields where used, the influence of the losses between the headgate and the heads of laterals being greater than has usually been supposed. Where canals cross gravel beds or gypsum deposits the results closely resemble trying to carry water in a sieve. Table No. 3 gives the depth of water used in irrigation, where the measurements were made at the canal headgates, and includes the loss from seepage and evaporation.

DIAGRAM SHOWING THE TIME OF IRRIGATION AND DEPTH OF WATER USED FROM THE GAGE CANAL IN DISTRICT NUMBER ONE, RIVERSIDE, CALIFORNIA.

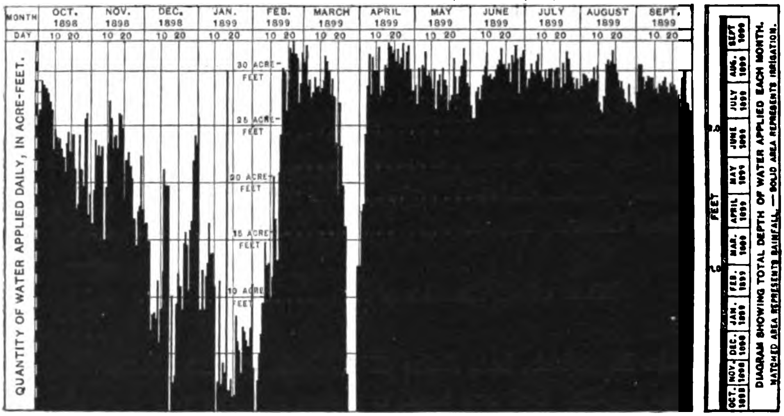


FIG. 9.

DIAGRAM SHOWING THE TIME OF IRRIGATION AND THE DEPTH OF WATER USED FROM THE BIG DITCH, SALT LAKE CITY, UTAH.

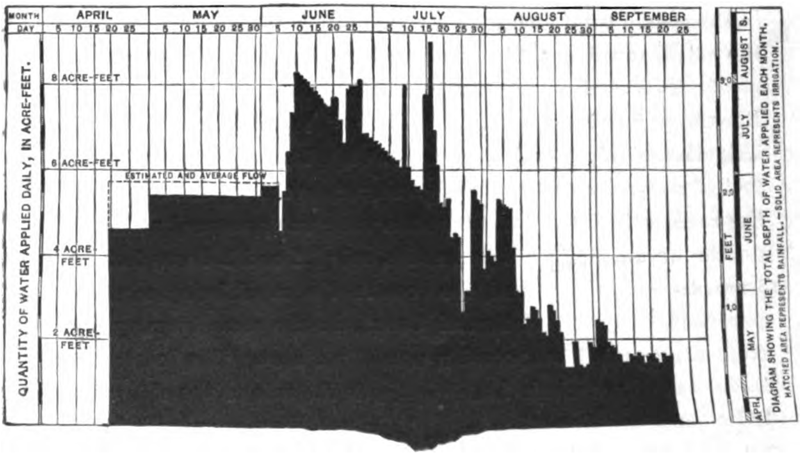


FIG. 10.

TABLE No. 3.—Depth of Water Used in Irrigation, when Losses in Main Canal are Included.

Name of Canal.	Depth of water used, in feet.
Pecos Canal, N. Mex	6.26
Mesa Canal, Ariz	3.81
Butler Ditch, Utah	6.24
Big Ditch, Utah	3.09
Upper Canal, Utah	6.30
Amity Canal, Colo	4.92
Rust Lateral, Idaho	5.06
Mean of all the above	5.10

For the Big Ditch, Salt Lake City, Utah, Fig. 10 shows when the water was applied and the total acre-feet applied per day, also the depth of irrigation water applied each month, and the rainfall for the same period.

A comparison of the duties in Table No. 3 with those obtained when the water was measured where used, will show that more than twice as much water was required where it was measured at the head-gate as where measured at the place of use. In other words, the losses in the canals from seepage and evaporation amount to more than half the entire supply. This is in accord with many of the measurements made on irrigation canals in India. Among those recorded in Buckley's "Irrigation Works in India" is one which shows that the irrigation of wheat under the Jamda Canal in Bombay required 5.6 acre-feet of water for each acre irrigated, where the water was measured at the head of the canal, but where the water was measured at the place of use it required, in two experiments, only 2.1 and 1.4 acre-feet, respectively, to irrigate an acre, the loss in the canal being more than 50 per cent. On the Hathmati Canal, in the same country, the loss from seepage and evaporation was 50 per cent. The losses in transit in our canals are much heavier than is the rule on the older canals of India, and are doubtless more general than they will be in this country when the banks of canals are older and when they are operated with greater regard for economy.

The reports of the special agents and observers in charge of these

measurements show that the losses from seepage and evaporation where measured in 1899 were as follows:

Pecos Canal, N. Mex., loss	52.3%
Mesa Canal, Ariz., estimated loss	25.0%
Middle Creek Ditch, Mont., loss in 4 miles	22.0%
With probable loss for entire ditch of	35.0%
Jordan and Salt Lake Canal, Utah, loss	45.0%
Gage Canal, Cal., loss	0.5%

The Gage Canal, being cemented, the losses from seepage are practically nothing. As compared to losses varying from 25 to 75%, shown in other canals, the loss of only half of 1% in this canal has great significance. The water turned into the head would have served to cover the land irrigated to a depth of 2.24 ft., while the mean depth for the water delivered to irrigators' laterals was 2.11 ft., a loss of only 0.13 acre-foot per acre irrigated. Canals can only be cemented on earth, as is done in California, in localities where frosts in winter are not severe.

Relation of Losses in Transit to the Amounts of Appropriations.

This loss from canals has given rise to a number of perplexing questions regarding appropriations. The states of Nebraska, Wyoming and Idaho have fixed a maximum limit on the amount of water per acre which an appropriator can acquire. In two of these states no one is permitted to appropriate more than 1 cu. ft. per second for each 70 acres irrigated. This limitation has given rise to the question as to whether the water so appropriated is to be measured at the margin of the irrigated field or at the head of the canal. Appropriators have claimed that if measured at the headgate the losses in the canal will be so great that the amount delivered will be inadequate for their needs; while water commissioners have insisted that it will be practically impossible to measure each appropriation at the head of the user's lateral. Several contests over this matter have already arisen, and sooner or later an authoritative decision will have to be reached. In one instance all the water entering the canal is delivered at a point 11 miles below. Measurements were made to determine the loss in this distance, and it was found to vary from 18 to 36 per cent. It was urged that this loss was so great that the amount turned into the canal should be increased enough to compensate therefor, but the State authorities refused to

recognize this claim, on the ground that such a concession would put an end to the improvement of ditches, since the greater the loss, the larger the appropriation which would be secured, and that its practical effect would be to place a premium on poor construction and wasteful operating of canals.

Seepage Waters Reappearing in Springs and Ditches.

Much of the water which escapes from canals finds its way to the surface below in the form of springs in what were originally dry ravines. Irrigators have filed on these springs, and secured thereby an ample water supply without having to pay the canal company which furnishes it anything for operating expenses or for the purchase of a water right. On the South Platte River alone there are over 400 of these filings on seepage waters. The report of the State Engineer of Colorado for 1898 shows that 5 000 acres in the Poudre Valley were irrigated with seepage water in that year. In a number of instances canal companies have sought to establish a title to the water of these springs and to collect for its transportation from their users; but the decisions of the courts in these cases have been conflicting, and no settled policy has as yet been established.

In some cases, where slopes are crossed by several canals, the lowest one frequently is benefited rather than injured by filtration, as it intercepts the water lost above. In one instance, where it is known that a large volume of seepage water is escaping from high-line canals, a ditch has been cut parallel with the river bank, but some distance away from it, to intercept this percolating supply. This has led to litigation to determine whether or not this is an interference with the rights of prior appropriators below on the main stream.

The percolating waters from canals and irrigated fields materially increase the water supply of western rivers. Measurements of this return or seepage water have shown that this reaches in many instances 30% of the original volume. In some cases it exceeds the original supply.

Influence of Fluctuations in Supply on the Duty Obtained.

A low duty does not of necessity indicate a wasteful or unskilful use of water. An illustration of this is found on streams which furnish more water than can be used during the flood season, but where the

period of plenty is followed by an equally assured period of drought. Irrigators have learned to provide against the latter by pouring on their land all the water it will hold while it can be had. By thus saturating the subsoil they store up a reserve supply which plants draw upon when the ditches fail. The report of one special agent deals with this practice, and a study of the flow of water in the Mesa Canal shows how marked is the scarcity during the hottest part of the year, and that at the time when the most water would be used if it could be had, less was actually used than at any other time in the season. That it is better to waste water on the land when there is a surplus, than to let it escape down the river and have crops burn later in the season, is beyond question, but the results of such irrigation are not nearly so satisfactory as they would be if this flood supply could be stored in reservoirs and be available for use when needed. Mr. Code's report shows clearly the necessity of reservoirs in localities like the Salt River Valley. They are needed to store the floods which now run to waste. They are needed to enable farmers to use water when crops demand moisture, and to relieve them from the alternating floods and drouths which dependence on the stream alone renders inevitable.

The Difference Between the Assumed Duty in Canal Contracts and the Duty Found by the Year's Measurements.

A comparison of the results of the year's measurements with the duty of water assumed in many important water-right contracts is interesting as showing their agreement or departure from actual practice. The quantity of water agreed to be furnished by a number of canal companies is as follows:

Arizona.

Arizona Water Company sells perpetual water rights, for 0.83⅓ cu. ft. per second to be used on not to exceed 80 acres. This gives a duty of a little less than 100 acres per cubic foot per second, or a depth of 7.5 ft. on the land irrigated.

Consolidated Canal Company sells perpetual water rights for not to exceed ⅓ miner's inch per acre. This gives a duty of 120 acres per cubic foot per second, or a depth of 6 ft.

Rio Verde Canal Company sells water by quantity, agreeing to furnish sufficient water to cover land to a depth of 2 ft. if it is called for, and more at the option of the company.

California.

Gage Canal allows 1 in. to 5 acres, or 1 cu. ft. per second to 250 acres. This water is not delivered in continuous flow, but in large streams for short periods, at the convenience of the consumer. This flow gives a depth of 2.89 ft.

Colorado.

Larimer County Ditch sells water rights for $\frac{1}{600}$ of the capacity of the ditch, without specifying how much land is to be irrigated.

New Loveland and Greeley Irrigation and Land Company sells one water right for each 80 acres, allowing a flow of 1.44 cu. ft. per second. The Colorado law compels companies to furnish water from April 1st to November 1st. For that length of season 1.44 cu. ft. per second would cover 80 acres to a depth of 5.3 ft.

Platte Valley Irrigation Company sells a water right for each 80 acres, allowing a flow of 1.44 cu. ft. per second, or a depth of 5.3 ft.

Fort Morgan Land and Canal Company sells a water right for each 80 acres, allowing a flow of not to exceed 1.44 cu. ft. per second for the irrigating season, or a depth of 5.3 ft.

Arkansas River Land, Reservoir and Canal Company sells a water right for each 80 acres, allowing a flow of not to exceed 1.44 cu. ft. per second, or a depth of 5.3 ft.

Dolores No. 2 Land and Canal Company sells water by the cubic foot per second, without limiting the consumer as to the area which he may irrigate.

Idaho.

Phyllis Canal contracts provide that the amount of water delivered shall not at any time exceed an amount equivalent to 1 cu. ft. per second for 50 acres, and that the total maximum quantity allowed shall not exceed 2 ft. in depth on the land irrigated; since 1899, will sell water by the cubic foot per second, with no limitations as to the area to be irrigated.

Boise and Nampa Canal, until 1899, delivered water at the rate of 1 miner's inch to the acre, but not to exceed 3 ft. in depth on the land irrigated. Since 1899, sells water by the cubic foot per second, with no limitations as to the area to be irrigated.

Montana.

Minnesota and Montana Land and Improvement Company sells water by the miner's inch without regard to the area to be irrigated. For the season of 1899 the farmers under this company's canal ordered, on an average, 1 miner's inch for 3.77 acres, showing a duty of about 150 acres per cubic foot per second.

Nebraska.

North Platte Irrigation and Land Company sells water rights for 1.44 cu. ft. per second, and prescribes land on which it shall be used.

Interstate Canal and Water Supply Company (Wyoming and Nebraska) agrees to furnish 1 in. per acre, or 1 cu. ft. per second for 50 acres. The legal season in Nebraska is 200 days—from April 15th to November 1st. One cu. ft. per second will cover 50 acres to a depth of 7.9 ft. in that time.

New Mexico.

Pecos Irrigation and Improvement Company water-right contracts provide for the delivery of 43 560 cu. ft. of water per acre, sufficient to cover the land to a depth of 1 ft., delivered at such times and in such quantities as may be necessary for the proper irrigation of crops.

Texas.

T. C. Purdy's water-right contracts call for the delivery of 43 560 cu. ft. per acre, to be delivered in not more than five irrigations per annum.

Washington.

Yakima Investment Company contracts to deliver not to exceed 1 cu. ft. per second per 160 acres, from April 1st to October 31st. This gives a depth of 2.65 ft.

Wyoming.

Wyoming Development Company sells water rights, giving the right to part of the total flow of canal.

Fetterman Canal Company sells a water right for each 8 acres, allowing a flow of $\frac{1}{10}$ cu. ft. per second. Wyoming canals do not ordinarily run more than 60 days. In that time this flow would give a depth of 1.46 ft.

Cody Canal shares represent water for 40 acres, and the quantity delivered is not to exceed $\frac{1}{2}$ cu. ft. per second. This gives a depth of 1.46 ft. in 60 days.

NEED OF CONTINUING THE INVESTIGATION.

The investigations of the past year did not deal with all the factors which influence the duty of water, because time did not permit of their study. For these reasons, the necessity for storage reservoirs has been left for subsequent study and discussion. The subject is

of commanding importance, because it cannot be considered apart from the fundamental question of who is to own or control the stored water, and likewise the stream from which it is taken or along which it is impounded. Sooner than is generally realized, the public or private ownership of western rivers is destined to be one of the great social and industrial questions of this country. Their waters are worth more than the land, public or private, through which they flow, and the manner of their disposal will do more to shape western civilization and promote or retard western development than all other causes combined. The study of the duty of water is a study of the farmer's needs, and it is hoped that the presentation of these needs will tend to promote the creation of an irrigation system which will make the supplying of his necessities of first importance, and be a matter of just pride to the nation.

TABLE No. 4—SUMMARY OF MEASUREMENTS OF DUTY OF WATER, 1899.

Station.	Period during which water was used.	GENERAL DUTY. Rainfall.	Evaporation.	Depth of irrigation.	Depth of irrigation and rainfall.	SPECIAL MEASUREMENTS OF DUTY. Location.	Depth of irrigation.	Depth of irrigation and rainfall.	Remarks.
Carlsbad, N. Mex.—		Feet.	Feet.	Feet.	Feet.		Feet.	Feet.	
Pecos Canal	Entire year	*0.31	4.55	6.26	6.57	Hagerman farm	15.44	15.75	Measurements carried on from April to October.
Mesa, Ariz. (Mesa Canal)—									
1896	"	0.78	....	5.91	6.69				
1897	"	1.04	....	5.55	6.59				
1898	"	0.57	....	5.01	5.58				
1899	"	0.39	....	3.81	4.20				
Riverside, Cal.—						District No. 1	2.32	2.79	
Gage Canal	"	0.47	....	2.24	2.71	District No. 2	2.23	2.70	
						District No. 3	1.78	2.25	
Salt Lake City, Utah—									
Butler Ditch	April–September	0.49	....	6.24	6.73				
Brown & Sanford Ditch	"	0.49	....	5.32	5.81				
Upper Canal	"	0.49	....	6.30	6.79				
Green Ditch	"	0.49	....	4.52	5.01				
Lower Canal	"	0.49	....	2.83	3.32				
Big Ditch	"	0.49	....	3.09	3.58				
Logan, Utah—									
Logan and Richmond Canal	June–September	0.27	3.27	3.59	3.86	Cronquist farm	2.60	2.87	
Lamar, Colo.—									
Amity Canal	March–September	0.91	....	4.92	5.83	‡ Biles lateral	1.82	2.50	
Gothenburg, Neb.—									
Gothenburg Canal	June 7th–September 30th	1.26	....	2.57	3.83	§ Daggett farm	4.24	5.50	
Wheatland, Wyo.—									
Canal No. 2	June–August	0.37	1.26	2.53	2.90	J. lateral	....	....	Evaporation at Laramie, Wyo.
						Oats	1.55	1.92	
						Corn	0.70	1.07	
Boise, Idaho—						Rust lateral	5.06	5.28	
Boise and Nampa Canal	May–September	0.22	....	....	....	A. F. Long farm	2.40	2.62	Rainfall at Boise.
						Wilson orchard	1.48	1.70	" "
Bozeman, Mont.—									
Middle Creek Ditch	June 16th–Sept. 16th	0.42	†1.74	2.10	2.52	Bozeman Exp. farm.			
						Clover	1.02	1.46	Rainfall 0.44 ft.
						Pease	1.10	1.51	" 0.41 "
						Grain	1.98	2.40	" 0.42 "
						Barley	0.98	1.39	" 0.41 "
						Oats	1.53	1.91	" 0.38 "
						"	1.34	1.70	" 0.36 "
						"	2.16	2.52	" 0.36 "
						"	1.28	1.72	" 0.44 "

* May 1st to November 11th, 1899. † Evaporation from July 6th to September 30th. ‡ Water used under Biles lateral, April-September. Rainfall at Holly, Colo., 0.68 ft. § Rainfall, April to September. Water used after harvest.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

THE IMPROVEMENT OF A PORTION OF THE JORDAN LEVEL OF THE ERIE CANAL.

Discussion.*

By William B. Landreth, M. Am. Soc. C. E.

Mr. Landreth. William B. Landreth, M. Am. Soc. C. E. (by letter).—The problem presented in the improvement of the canal through the marl beds was that of deepening an existing channel and not of building a new one. Owing to the high and wide spoil banks formed by the material excavated in the first construction of the canal, the general plan proposed by Mr. Rafter, a wide channel with flat slopes, if adopted in the 1896 improvement, would have proven more expensive than the timber and pile construction.

No slides have occurred in any portion of the work described, where struts were placed across the prism. Several careful examinations of the timber construction, made by the writer during the past winter and spring, show that no movement of the struts or slope wall has taken place.

When the water was let into the canal in May, 1898, a portion of the tow-path bank, where only short piles had been used under the slope wall, slid into the prism. This slide occurred in front of the Empire Portland Cement Works, and was probably caused by the vibration of heavy engines and grinding mills in the works adjacent to the tow path. Bids were asked for in November, 1899, for the repairing of this slide, upon plans identical with the pile and timber con-

* Continued from April, 1900, *Proceedings*. See December, 1899, *Proceedings*, for Paper by William B. Landreth, M. Am. Soc. C. E., on this subject.

Mr. Landreth.

struction used in 1897–1898, but on more stringent specifications regarding the re-excavation of material.

After several competent contracting firms had examined the locality, plans and specifications, only one contractor submitted a bid. The contract was awarded to him, and the work has been completed in a satisfactory manner for 12% less than the engineer's preliminary estimate.

The price bid for bailing and draining on the last contract amounted to $12.50 per lineal foot of prism, and, taking the length of prism in the marl beds on the former contract as 2 000 ft., the bailing and draining on that contract would have cost $250 000, at the same rate per foot.

The 1899 work, done by contract, on the same plan as the 1897 work and under the direction of the same engineer, cost about 20% more per lineal foot than the 1897 work done under "force account."

The case of the Canandaigua Outlet, cited by Mr. Rafter, is not a parallel case with the marl bed work on the Jordan Level; the former being a dredged channel from which the water is never removed.

The drainage ditches, noted in Mr. North's discussion are of permanent value to the State by lessening the saturation of the canal banks and preventing the surface water from entering the prism.

A calculation of the quantities on Contract No. 4, based on the system of unit prices mentioned by Mr. North, shows that there would have been a difference of about 1% in favor of the contractor, provided the prices estimated and bid had been the same as by the old method.

The discussions on quicksand treat the subject from scientific and practical points and have added materially to the literature thereon, thus attaining one of the objects in view in the preparation of the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

PAPERS AND DISCUSSIONS.

HISTORY OF THE PENNSYLVANIA AVENUE SUBWAY, PHILADELPHIA, AND SEWER CONSTRUCTION CONNECTED THEREWITH.

Discussion.*

By GEORGE S. WEBSTER and SAMUEL TOBIAS WAGNER, Members, Am. Soc. C. E.

Messrs. Webster and Wagner.

GEORGE S. WEBSTER and SAMUEL TOBIAS WAGNER, Members, Am. Soc. C. E. (by letter).—It will be noticed by an examination of Fig. 4, which is the main drainage plan of the sewers, that the grades on the main lines are very light for the conditions which had to be met. In place of compensating for the velocity on the curves by increasing the grades, it was deemed expedient to keep the same rate of grade throughout and add to the velocity by reducing the friction. This was done by plastering the inverts on curves with Portland cement mortar ½ in. thick. This plastering was usually done after the invert was laid and before the centers for the arch were placed. In the case of Contract No. 1 of the Callowhill Street System the work of plastering was delayed until the sewage was passing through the system, with the result already referred to in the paper.

* This discussion (of the paper by George S. Webster and Samuel Tobias Wagner, Members, Am. Soc. C. E., printed in the *Proceedings* for February, 1900), is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to June 22d, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

All the brickwork of the sewers, except that in the wellholes, was laid in natural cement mortar. In the wellholes, Portland cement was specified. In all cases the mortar was mixed in the proportion of 1 part of cement to 2 parts of sand. Briquettes, 1 sq. in. in cross-section, when made from natural cement mortar from the mixing box, were required to develop an ultimate tensile strength of 40 lbs., after having been 1 day in the air and 6 days in water. In the case of Portland cement, under the same conditions, a strength of 150 lbs. was required. Messrs. Webster and Wagner.

The other requirements of the natural cement, which, of course, represented the greater part of the work, were as follows: It shall weigh not less than 112 lbs. per imperial bushel. The residue upon a No. 50 sieve shall not exceed 4% by weight, upon a No. 100 sieve 25%, and upon a No. 200 sieve 50 per cent. Pats of cement ½ in. thick, temperature between 60 and 70° Fahr., shall develop initial set in not less than 10 minutes and hard set in not less than 30 minutes, the amount of water used being just sufficient to form a stiff plastic paste.

The following tensile strengths were required:

24 hours (in water after hard set)	75 lbs.
7 days (1 day in air, 6 days in water)	150 "
28 days (1 day in air, 27 days in water)	250 "

Mortar composed of 1 part of cement and 1 part of standard quartz sand was required to develop an ultimate tensile strength of 75 lbs. after 7 days.

All concrete used was composed of 1 part of natural cement, 2 parts of sand, and 4 parts of stone or furnace slag.

The results of all soundings made adjacent to the line of the sewer were platted on the working drawings of the sewers, for the benefit of the contractors in making their estimates. The following note was placed upon all such plans: "Soundings shown are not guaranteed or binding upon the City of Philadelphia." As a result of this note no claims were made where any differences between the soundings and the workings were found to exist.

A summary of the character of the material encountered is approximately as follows: Beginning at Thirteenth St., where the depths of the sewers were smallest, the excavation was through clay and gravel. The invert of the sewer, however, usually rested upon rotten rock of a micaceous nature. The cross sewer on Fifteenth St. was mostly in gravel, and the workings naturally contained a considerable amount of ground-water. On Sixteenth St. a considerable amount of rather hard micaceous rock was met, the invert being entirely in this material. From this point to between Nineteenth and Twentieth Sts. clay, gravel and rock of varying degrees of hardness, irregularly located, was encountered. On Seventeenth and Eighteenth

Messrs. Webster and Wagner. Sts. very rotten micaceous rock, acting like quicksand, was discovered, requiring great care in the workings. Between Nineteenth and Twenty-second Sts., and on the Twentieth St. sewer, hard gneiss was found. As the work approached the river the rock gradually ran out, until, between Twenty-third and Twenty-fourth Sts., it had entirely disappeared.

On Twenty-fourth St. from Callowhill St. to Pennsylvania Ave. reasonably hard, and in some cases very hard, gneiss was met. Beneath Pennsylvania Ave. was a bed of gravel, afterward discovered to be of considerable extent; the sewer invert, however, was on rock. From this point to Twenty-ninth St. the excavation was through soft micaceous rock which blew to pieces and came out of the shafts to all purposes in the form of sand. From Twenty-ninth to Thirtieth St. the excavation was through clay and gravel, as the sewer was not deep.

The character of the rock east of Twenty-ninth St. was the same 40 ft. below the surface as immediately under the top soil.

The difficulty of making deep open-cut excavations adjacent to buildings, without causing settlement, is well known. This is especially true if the foundations of the buildings be inadequate, or be upon a formation which is readily affected by the action of the elements upon exposure, or if the buildings be old and poorly constructed. In many cases the trouble is aggravated if it is necessary to use an explosive to assist in the removal of the excavated material.

If, on the other hand, the excavation is made in tunnel, with careful timbering in soft material, or with properly regulated charges when blasting through rock, the danger is materially lessened, especially if all open spaces between the structure which is to be built and the surrounding material are compactly filled with masonry, carefully laid.

The construction of these sewers has been an excellent proof of the truth of this statement. Probably seven-eighths of the entire line was through streets lined on either side with buildings of all kinds, dwellings, store-houses and manufactories. In no case, where the sewers were constructed in tunnel, was any damage done to property, with the exception of the cases referred to in the paper. All such damage was caused by blasting, and, without doubt, was brought about by the use of unduly large charges. In the case mentioned in Contract No. 2, Pennsylvania Avenue System, a 48-in. water main, immediately over the tunnel, was broken by blasting, and an ice manufacturing plant, similarly situated, was compelled to stop work on account of the leakage of the joints in the ammonia pipes, which, it was claimed, was caused by the blasting. In this latter case it was the intention to remove the plant by widening the street, so that the damage was not unexpected. On Contract No. 2, Callowhill Street System, the only damage was caused by the interference with the adjustments of very

sensitive machine tools in an adjoining plant. In this case, further damage was prevented by the Chief Engineer, who gave instructions regulating the size of the blasts. The damage in this case, recovered in court from the contractor, amounted to $300, which was the full amount claimed by the plaintiff. In no case, where the sewers were in tunnel, was there any damage whatever caused by settlement, nor were any settlements in the street paving afterward discovered, except in a few cases over the points where shafts had been sunk in order to reach the tunnel. Messrs. Webster and Wagner.

The only open-cut work was in comparatively shallow depths. On Callowhill St., between Twenty-second and Twenty-third Sts., there was a slight settlement of the curb and part of the sidewalk pavement nearest to the trench. This was rectified by the contractor at a slight expense.

On Twenty-fourth St. considerable damage to adjoining buildings was caused by the construction of the temporary flume, because it was excavated through rock, and, also, because of the very old and dilapidated character of the buildings. The expense of repairing these buildings was borne by the contractor, as already stated. The location of the flume on the sidewalk was determined by the contractors as an incidental to the work.

The effect of the sub-soil drainage upon the surrounding neighborhood, caused by the construction of the sewers, has been marked. A very considerable amount of this water was encountered on the cross streets from Fifteenth to Eighteenth Sts., and was tapped into the sewers. When the excavation for the retaining walls was made, afterward, no trouble whatever was caused by water, in many cases the soil being perfectly dry where formerly running sand was encountered.

It is the purpose of the writers to present in a future paper a description of the construction of the Subway and Tunnel proper, in which the work of underpinning, the construction of the retaining walls and the bridges, as well as the special railroad and municipal features of the operation, will be elaborated.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

EXPERIMENTS ON THE FLOW OF WATER IN THE SIX-FOOT STEEL AND WOOD PIPE LINE OF THE PIONEER ELECTRIC POWER COMPANY, AT OGDEN, UTAH. SECOND SERIES.

Discussion.*

By Messrs. CHARLES W. SHERMAN and G. S. WILLIAMS.

Mr. Sherman. CHARLES W. SHERMAN, Jun. Am. Soc. C. E. (by letter).—Mr. Rafter's discussion on the effect of animal and vegetable life on the discharging capacity of a pipe is very interesting, but needs to be supplemented by records of actual observation of the presence or absence of such growths. The writer submits these few lines in the hope of adding his mite to the meager data on this subject.

Late in the year 1894, a 36-in. force main was laid from the Chestnut Hill Pumping Station to the Fisher Hill Reservoir of the Boston Water-Works, the distance being something over a mile. Early in the following year, during a test of the new Leavitt pumping engine, observations of the loss of head were taken at three points in a length of about 5 000 ft., from which it was found that the coefficient c of the Chezy formula was 136, v being 4.7 ft. per second, or about what would be expected for a new cast-iron pipe of this size. A year later, or early in 1896, an extensive series of experiments on the friction

*Continued from April, 1900, *Proceedings*. See February, 1900, *Proceedings*, for paper by Charles D. Marx, M. Am. Soc. C. E.; Charles B. Wing, Assoc. M. Am. Soc. C. E., and Leander M. Hoskins, C. E., on this subject.

loss in this pipe was made by the writer, under the direction of Desmond FitzGerald, Past-President, Am. Soc. C. E., with velocities ranging from 1.1 to 4.5 ft. per second. It was supposed that the pipe was in about the same condition, but the results showed a coefficient of about 113, a great loss within a single year. This result was so far from that expected that the pipe was partly drained, and entered at the upper end. Only a small amount of tuberculation was found, but the whole interior surface was covered with a slimy substance which proved upon examination to consist almost wholly of the polyzoon *Fredericella*. In 1897, another experiment, in which v was 3.2 ft. per second, showed c to be about 114. It thus appears that there was a great change in the capacity of the pipe in the first year after it was laid, and practically none in the year following. Mr. Sherman.

Late in 1897, this pipe was cut into, for the purpose of making connections near the pumping station, and an examination of the interior surface at this point showed that practically the same conditions obtained. If anything, the organic growth was somewhat thicker, as was to be expected, this being nearer the source of the food supply.

It seems to the writer that, with New England surface waters, which contain more or less organic matter, such as algæ, to furnish food for them, growths of polyzoa on the interior of the pipes are to be expected, and the result will be a large diminution of the capacity of the pipes within the first year. After that there will be a further gradual diminution of capacity due to the slow increase of tuberculation. With ground-waters or filtered waters, which have not been exposed to the light, such growths will probably not occur, as food for the polyzoa will be lacking. The experience in Brookline, which has a ground-water supply, and where such growths have not been observed, seems to bear out this opinion.

GARDNER S. WILLIAMS, M. Am. Soc. C. E. (by letter).—On page 506 of the writer's discussion in the April number of *Proceedings*, the coefficient of the equation in the third paragraph has been erroneously computed. The equation in its correct form would be $H_f = 2.8643\ H_v$. This changes Fig. 4, Plate XXIII, by making the equation showing increased loss of head due to tunnel $= H_f = (8.74 - 8.25)\ H_v = 0.49\ H_v$, whence the increased loss of head in Sections 3–4 and 4–5 combined, due to the presence of the tunnel, amounts to about 6%, not 60% as incorrectly stated on page 508. These are errors for which the writer alone is responsible. Their discovery came too late for correction in the April number of *Proceedings*. Mr. Williams.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

THE PRACTICAL COLUMN UNDER CENTRAL OR ECCENTRIC LOADS.

Discussion.*

By ERNST F. JONSON, Assoc. M. Am. Soc. C. E.

Mr. Jonson ERNST F. JONSON, Assoc. M. Am. Soc. C. E. (by letter).—The author states that the curve of flexure of a column of uniform cross-section and elasticity is a curve of sines only when the eccentricity of loading is infinitely small.

The writer, therefore, would call attention to the fact that the nature of the curve is in nowise changed by increasing the eccentricity. Considering the line in which the external forces act, as the axis of abscissas, the bending moment is still proportional to the ordinate. Hence, no matter how great the eccentricity may be, the curve is a curve of sines, the equation of which is

$$y = D \cos. \frac{x}{r}\sqrt{\frac{w}{E}} \qquad \text{I}$$

where x is abscissa, y ordinate, D maximum ordinate, r radius of gyration, w load per unit of cross-section area, and E modulus of elasticity.

For maximum w

$$D = \frac{r^2\ s}{a\ w} \qquad \text{II}$$

*This discussion (of the paper by J. M. Moncrieff, M. Am. Soc. C. E., printed in the *Proceedings* for March, 1900) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to June 22d, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

where a is distance to extreme fiber and $s = p - w$ when $p - w \overline{\overline{<}}$ Mr. Jonson.
$t + w$, or $= t + w$ when $p - w > t + w$, p being the proportional limit in compression and t the proportional limit in tension.

Hence,

$$y = \frac{r^2}{a} \frac{s}{w} \cos. \frac{x}{r} \sqrt{\frac{w}{E}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{III}$$

If y is given, the value of $e = c + b$, c being the equivalent eccentricity, and b the intentional eccentricity, x becomes the length of the elementary column, *i. e*, the column fixed at one end and free at the other. We have then

$$c + b = \frac{r^2}{a} \frac{s}{w} \cos. \frac{l}{r} \sqrt{\frac{w}{E}} \ldots\ldots\ldots\ldots\ldots\ldots \text{IV}$$

or

$$l = r \sqrt{\frac{E}{w}} \cos.^{-1} \frac{a}{r^2} \frac{w}{s} (c + b) \ldots\ldots\ldots\ldots\ldots\ldots \text{V}$$

It seems to the writer that this more correct formula is just as simple as the one proposed by the author.

Apart from its mathematical form the foregoing formula differs from that of the author in that it makes the maximum load a function, not of the ultimate strength of the material, but of the proportional limit. When this limit has been passed at the point of greatest bending the deflection increases very rapidly, owing to the decrease of the modulus of elasticity. If the eccentricity is very small a very small addition to the load will produce an infinite deflection. If the eccentricity is larger, the extent to which the column may be strained beyond the proportional limit also becomes larger. Until this point has been determined we must, however, regard the proportional limit as the determining factor in calculating the strength of columns.

That the strength of centrally-loaded columns is quite independent of the ultimate strength of the material has been experimentally demonstrated by M. Considère. He found that columns of steel, with a proportional limit of 71 000 lbs. and an ultimate strength of 88 500 lbs. per square inch, were over 10% stronger than columns of steel, which had a proportional limit of 64 000 lbs. and an ultimate strength of 98 000 lbs. per square inch.

From the fact that the deflection of a column is proportional to the eccentricity of the load, other things being equal, the author argues that in designing a column it is necessary to guard against two totally independent modes of failure, viz., failure by excessive intensity of fiber stress, and failure by instability. He accordingly proposes the use of two formulas (7 and 2), or rather two forms of one formula, the second being nothing but the special form which the first assumes when the equivalent eccentricity is reduced to zero. This is clearly unnecessary. If a column is strong enough to sustain a certain load

Mr. Jonson. eccentrically applied, it is also strong enough when this load is applied exactly in the axis of the column.

In Fig. 46, the curve of Formula 2 falls below that of Formula 7 when $\frac{l}{r} > 88$. But this is due to the application of a factor of safety to E in the former formula. If the same factor of safety had been applied to E in both formulas, Curve 2 would always have been above Curve 7.

The formula here given was, as far as the writer knows, first applied to experimental results by A. Marston, Assoc. M. Am. Soc. C. E. Mr. Marston, the same as the author, proposes a constant value for the relation $\frac{a\,c}{r^2}$. This implies that a short column is more likely to be imperfect than a long one.

It seems to the writer that the amount of imperfection in a column ought to be proportional to the length, hence, since the effect of imperfection is also as the length, the equivalent eccentricity ought to be proportional to the square of the length of the column.

Now, if l^2 were introduced into the cyclometrical function of the formula, it could not be solved conveniently. We will, therefore, substitute the value $\frac{1}{w}$, thus making $c = \frac{r^2\,k}{a\,w}$, where k is the empirical constant, the coefficient of imperfection, and consequently Equation V becomes

$$l = r\sqrt{\frac{E}{w}}\ \cos.^{-1}\left(\frac{k}{s} + \frac{a\,b\,w}{r^2\,s}\right) \ldots\ldots\ldots\ldots\ \mathrm{VI}$$

How this conclusion agrees with experience, may be seen from Table No. 5, which shows the coefficient of imperfection of some of the round-ended wrought-iron columns tested by Mr. James Christie. This table seems to show that we come much nearer the truth by assuming k to be a constant than when we make it a variable, proportional to w. A very close agreement, of course, cannot be expected.

TABLE No. 5.

No.	$\frac{l}{r}$	w.	k.	Averages.
221	44	37 148	562	
222	44	32 000	4 430	
220	63	32 900	2 170	3 928
214	80	24 138	8 209	
219	81	25 417	4 270	
205	129	13 482	3 772	
216	138	13 112	1 901	
201	179	7 874	2 160	2 506
207	187	7 239	2 193	
202	229	4 863	2 159	2 159
228	305	2 740	2 244	3 435
208	306	2 476	4 627	

$k = (p - w) \cos. \frac{\quad}{2\,t}\sqrt{\frac{\quad}{E}}$, $p = 38\,000$, $E = 28\,000\,000$.

It seems to the writer that a fractional factor of safety should be applied to each of the three empirical constants in the formula, viz., E, k and p or t. Designating this factor by K, the least of the two values $K_3\,p - w$ and $K_3\,t + w$ by s_1, we have the formulas for columns of uniform cross-section and elasticity in their final form.

One end fixed and one end free:

"Central" load;

$$\frac{l}{r} = \sqrt{\frac{K_1\,E}{w}} \cos.^{-1} \frac{k}{K_2\,s_1}$$

Eccentric load;

$$l = r\sqrt{\frac{K_1\,E}{w}} \cos.^{-1}\left(\frac{k}{K_2\,s_1} + \frac{a\,b\,w}{r^2\,s_1}\right)$$

Both ends hinged:

"Central" load;

$$\frac{l}{r} = 2\sqrt{\frac{K_1\,E}{w}} \cos.^{-1} \frac{k}{K_2\,s_1}$$

Eccentric load;

$$l = r\sqrt{\frac{K_1\,E}{w}}\left[\cos.^{-1}\frac{k}{K_2\,s_1} + \cos.^{-1}\left(\frac{k}{K_2\,s_1} + \frac{a\,b\,w}{r_2\,s_1}\right)\right]$$

One end fixed and one end hinged:

"Central" load;

$$\frac{l}{r} = 3\sqrt{\frac{K_1\,E}{w}} \cos.^{-1} \frac{k}{K_2\,s_1}$$

Eccentric load;

$$l = r\sqrt{\frac{K_1\,E}{w}}\left[2\cos.^{-4}\frac{k}{K_2\,s_1} + \cos.^{-1}\left(\frac{k}{K_2\,s_1} + \frac{a\,b\,w}{r^2\,s_1}\right)\right]$$

Both ends fixed:

$$\frac{l}{r} = 4\sqrt{\frac{K_1\,E}{w}} \cos.^{-1} \frac{k}{K_2\,s_1}$$

Both ends flat:

$$\frac{l}{r} = 4\sqrt{\frac{K_1\,E}{w}} \cos.^{-1} \frac{k}{K_2\,w}.$$

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PAPERS AND DISCUSSIONS.

RECENT STADIA TOPOGRAPHIC SURVEYS: NOTES RELATING TO METHODS AND COST.

Discussion.*

By Messrs. EMIL LOW, JOHN F. WALLACE, HENRY B. MAGOR, R. S. BUCK, HENRY GOLDMARK, A. J. HIMES, R. A. MACGREGOR, KENNETH ALLEN and WAGER FISHER.

Mr. Low. EMILE LOW, M. Am. Soc. C. E. (by letter).—This paper recalls to the writer two topographical surveys made by him several years ago. One, a survey of Prince's Flats and vicinity, Wise County, Va., and the other, a portion of the property of the Mathieson Alkali Works, at Saltville, in Smyth and Washington Counties, Va.

Prince's Flats is now called Norton, and is the junction of the Norfolk and Western and Louisville and Nashville Railroads. It comprises a stretch of land between the Goest's and Powell Rivers, which streams rise to the north of the town, flowing southward almost parallel to one another only a few miles apart, and at Norton turn, the former flowing to the east, the latter to the west.

At the time of the survey, part of Prince's Flats consisted of a little cleared land, the remainder and the adjoining country being a virgin forest and a howling wilderness. The Norfolk and Western Railroad Company had obtained an option on the land, which had been selected for a town site and junction point. Before acquiring the

* This discussion (of the paper by William B. Landreth, M. Am. Soc. C. E., printed in the *Proceedings* for March, 1900) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to June 22d, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

land the company desired to have a topographical survey made, and this duty was assigned to the writer. The option had only a short time to run, and it became imperative to make the survey and map in the shortest possible time. The survey was made in one month by a small party organized for this purpose and consisting of an assistant engineer, transitman, chainmen, back flag and axemen. Mr. Low.

The area to be surveyed was about 2 miles long and nearly ½ mile wide, and consisted of several tracts of land, on one of which the boundary line had been run out by a local land surveyor. The territory was quite rugged, the difference in elevation between the lowest and highest points being about 800 ft. Many of the slopes ranged from 30 to 45 degrees.

A wagon road ran through the property from one end to the other, cutting it into two nearly equal parts, and it was decided to use this for the location of a base line. A transit line, marked by stakes at 100-ft. intervals, was run along the road, and from this base line, especially on the north side, spur lines were run up the various water courses and also along the tops of ridges and crests. On the south side additional base lines were run, generally parallel with the main base. The boundary lines were also re-run, and on all lines stakes were placed at 100-ft. intervals. Nearly all the lines were carefully levelled over with a wye level, and elevations of all stations were taken, as well as of intermediate points, where necessary. A few of the auxiliary lines were run by a second party, which happened to be on the ground at the time, making surveys for the proposed railroad.

After the levels had been run, the topography was taken. This included all the natural as well as the few artificial features which existed, also the contours, which were taken at 5-ft. intervals.

The contours were taken by means of a hand or Locke level, the actual elevation ending in 0, and 5 ft. being ascertained on the ground and the distances from the center line noted. All measurements and observations were taken on lines at right angles to the base line. Some of the cleared areas were surveyed by the stadia method, in much the same manner as outlined in the paper.

The work of running the base lines, taking the levels of the same and the topography, so-called, was all done by one and the same party, each part of the work being taken up in regular sequence. The plotting was not done in the field, but at a neighboring farm house, on an improvised drawing table, after the base lines had been run, and while the levelling and topographical work was in progress.

The methods used in this survey gave very close results, it being especially desired that the location of the contour lines should be quite accurate, it being the intention to utilize the survey in laying out the town site, later on. A less accurate survey would have necessitated a second survey as a basis for laying out the streets. The

Mr. Low. streets were laid out several years after the survey was made, and it may be of interest to note, that very few, if any, changes or variations from the paper projection were required, showing remarkable accuracy in the contour work, which is especially noteworthy, when it is recalled that the area was densely wooded and covered with a thick growth of underbrush and laurel. The surveyed area comprised about 2 sq. miles, and the cost of the survey approximated $600, or $300 per square mile.

The other survey was that of a portion of the lands of the Mathieson Alkali Works, at Saltville, Va. This land formerly belonged to the Holston Salt and Plaster Company, a corporation engaged in the manufacture of salt. The Mathieson Alkali Works secured the property for the purpose of erecting thereon a plant, for the manufacture of soda-ash and allied products. The contemplated works were to be very extensive and to employ a large force of men. As the old company had no maps of their property, on which the various salt furnaces, salt wells, houses and other improvements were located, the new company decided to have a complete topographical survey made, and this duty devolved upon the writer.

The survey generally comprised the valley of the Mill Spring Branch, as the stream was called locally. In this valley were situated the salt wells, and along the edges the various improvements, the whole valley being rimmed in by hills nearly 300 ft. high. The Saltville Branch of the Norfolk and Western Railroad skirted the west side of the valley. The survey also included part of the valley of the Holston River, on the banks of which the works of the company are located.

In this case, also, it was desired to have an accurate survey of the property, and especially one which would show the true location of the contour lines. It was an ideal piece of country for a stadia survey, being entirely devoid of timber, the land not occupied being used for grazing cattle. Owing to the many artificial features involved, it was not considered advisable to use the stadia method, but to use instead that generally adopted on railroad surveys.

As at Prince's Flats (or Norton), base lines, marked by stakes set every 100 ft., were run in suitable and selected places. For instance, one was run up the middle of the valley, another along the railroad track, and others were run along the different highways, along the foot of the hills which skirted the valley, some higher up the hill sides, and some along the tops. Care was taken to run lines wherever possible along the ridges or high ground and as nearly as possible at right angles to the slopes. Spur lines were run where necessary along small streams and up ravines. All lines were run with a transit, and, in all but a few instances, formed closed circuits, which usually closed exactly or with a variation of a few minutes only. Each line was

marked with a distinctive letter, for the purpose of identification on the ground. Mr Low.

Levels were run over these lines, the elevations of all stations being taken. The levels checked closely, the variations not exceeding a few hundredths of a foot.

The topography was all taken by the offset method, that is, on lines at right angles to the bases, and at the regular stations, placed, as stated, at 100-ft. intervals, and in a few instances at 50-ft. intervals.

All buildings of every kind were located, two measurements to each being taken, one the distance from the base line on range of one of the sides of the building to the corner, and the other from the same corner to the nearest station, the station and plus, where the range cut the center line, being, of course, also noted. The size, character, etc., of the building was also noted. The contours were taken, as at Prince's Flats, by a hand level.

The maps were plotted to a scale of 200 ft. to 1 in., or $\frac{1}{2400}$. The map sheets were 19 x 24 ins. in size, fitted together at the adjoining edges by crosses. The lines were plotted by means of a large paper protractor, using a standard meridian, to which the lines were referred. The main base line, extending from the Holston River to Buena Vista, a distance of about 3 miles was checked carefully by means of latitudes and departures. As nearly all the other lines were on closed circuits, tied to the main base, the plotting could be checked in those instances, and it usually closed remarkably well. Upon the completion of the maps, a tracing of the whole area was made.

The area surveyed was about 3 sq. miles. The cost was about $900, or $300 per square mile. This is hardly a fair way to express the cost, as the areas were comparatively small, and, also, on account of the large amount of detail taken, which is not the case in making surveys covering large areas, devoid of many artificial features, where only salient points are taken and the intervening portions sketched in or interpolated.

John F. Wallace, President, Am. Soc. C. E.—The speaker was, for a short time, Vice-President and Manager of the Mathieson Alkali Works. One of the first pieces of information which he desired was a map showing the location and physical features of the property, and the results of Mr. Low's topographic survey were brought to him. Mr. Wallace.

The company owned from 13 000 to 14 000 acres of land, and had established, on the banks of the Holston River, a plant for the manufacture of soda-ash and its allied products. The total amount invested in plant and machinery was about $3 000 000.

The works were laid out originally by an English engineer, who had carried out the idea of the English industrial village. Each class of employees had its own settlement. The superintendents of departments had one section of a valley to themselves; the carpenters and

Mr. Wallace. the foremen of certain departments had another valley; the laborers in certain parts of the process still another valley; and the ordinary laborers another. The location of the buildings, the side-tracks, the drainage and other improvements, which were being considered and discussed continually, made a map of this kind an essential feature of the economical management of the property.

For the area covered, the speaker has never known of a more complete or accurate topographic survey than that by Mr. Low. He has had occasion to test it in innumerable ways, and, to his pleasure, has never found any errors. In the consideration of various projects for the development of the property, the location of roads, side-tracks, trestles, and apparatus to take care of the waste products, etc., he was able to make profiles in any desired direction, and the assistance given by the information embodied in this survey was so great that the speaker has always felt indebted to Mr. Low for it, and takes this opportunity to pay him a tribute. A great many engineers, like Mr. Low, spend their lives in obscure sections of the country, and we never hear of them. The faithfulness with which they carry out their work is never known. It is with pleasure, therefore, that the speaker calls attention to the accuracy of this particular survey.

Mr. Magor. HENRY B. MAGOR, Jun. Am. Soc. C. E.—The speaker was engaged in making the survey for the Hankow and Canton Railway, in China, of which William Barclay Parsons, M. Am. Soc. C. E., was chief engineer. Before sailing for China it was a question as to what kind of a survey, or, in fact, whether any survey whatever, could be made. Mr. Parsons, however, decided that a stadia survey was the only one practicable. The party consisted of a chief of party, a topographer, an instrument man and two rodmen.

Before leaving Hankow it was considered absolutely necessary to check all the instruments and measure the rods. The latter was done by laying out a long base line on the Bund (the principal street in Hankow). With the assistance of some of the native policemen, and with three days' work, the subdivision of the rods was made exactly right. This was an important preliminary as the readings on the survey were not to be checked, and if the rods had not been correct there would have been a large cumulative error in the total length of the survey, 742 miles.

Stadia readings were taken for the distances, and the levels were taken by stadia readings and vertical angles, and the courses from the compass. The distance was covered in 75 working days, making an average of about 10 miles a day.

At the beginning of the survey a sight was taken on a pagoda, the latitude and longitude of which had been accurately determined to seconds by the Customs Department. At the end of the survey a sight was taken on another pagoda, the location of which had also

been similarly determined. The latitudes and departures were then worked out and the total distance checked within less than 2 000 ft., or less than ½ mile in 742 miles, which shows how accurate a stadia survey can be made. Mr. Magor.

The sights averaged about 1 500 ft., but one (when it was necessary to get away from a crowd in a hurry) was as great as 5 500 ft. Whenever it was possible, solar observations were taken to determine the declination of the needle. The first reading was 45 minutes west, and zero, or the magnetic meridian, was passed when about three-quarters of the distance had been traversed. Thus, fortunately, there were practically no corrections to be made in reckoning the latitudes and departures.

The levels, of course, could not be expected to check, in fact, no attempt was made to check the readings for elevation. The line was merely run from one point to another, the object in taking any levels whatever was to determine any particular gradient over a summit, and this, of course, was accomplished satisfactorily. The levels, however, checked reasonably well with the barometric readings on the summit, which was about 1 000 ft. above the sea; but neither of these results could be relied upon, as they were only approximate.

The stadia rods were 15 ft. in length, and in measuring very long distances half the wire interval was used and then the remainder was determined by the gradienter. Corrections of the distance for differences in elevation were not made. At certain places where the vertical angle was great, the correction of the distance was computed, but was so small that it would have been of no importance on a survey of this kind.

R. S. Buck, M. Am. Soc. C. E.—In the Government survey of the Red River it was endeavored to limit the length of the sights to about 600 or 700 m. The longest sight was about 1 000 m., but in such long sights there was a likelihood of considerable error. Mr. Buck.

All the speaker's stadia work has been done in level country, and he would like to inquire as to what extent corrections for distance are applied in making surveys in very hilly country.

Henry Goldmark, M. Am. Soc. C. E.—In the work in the Rocky Mountains, with which the speaker was connected, the stadia was only used to a small extent. John Q. Barlow, M. Am. Soc. C. E., did the instrumental work and made some very rapid surveys by that method He also surveyed some of the auxiliary reservoirs (not yet built) of the Ogden power plant by stadia, but the principal reservoir site was surveyed by the ordinary railroad methods, that is, by base lines and by secondary lines at right angles to these bases, all elevations being taken with a level. Mr. Goldmark.

There is no doubt that the stadia method would have been much more rapid and economical than the method adopted for that survey,

Mr. Goldmark. and would have been sufficiently accurate. The surveys were very good, but the speaker has always considered them as more expensive than was necessary.

Mr. Himes. A. J. HIMES, M. Am. Soc. C. E.—In making the United States Deep Waterways Surveys for a 30-ft. canal along the Oswego-Mohawk route, the territory was divided into two parts, known as the Eastern and Western Divisions, and the survey of each part was made by a separate corps of engineers. The dividing line was at Herkimer, a few miles west of Little Falls.

The route, on the Western Division, followed the Oswego River, Oneida Lake, Mohawk River and connecting lines, a total distance of about 91 miles. The work on this division included the necessary harbor surveys at Oswego, soundings in Oneida Lake and the various streams, the triangulation of Oneida Lake, covering an area of about 78 sq. miles, a complete system of test borings and about 120.7 sq. miles of topography. The topography was taken with the stadia, in accordance with instructions issued by the United States Board of Engineers on Deep Waterways.

A base line was measured along the route, and Oneida Lake was covered with a chain of quadrilaterals. This base line and triangulation formed the foundation of the survey.

Method.—The manner of taking the topography was: To start from an instrument point on the base line, with the base-line azimuth, and run a circuit through the territory to be covered, returning, generally within a distance of 2 miles, to the base line and checking the azimuth on the nearest base line point.

The courses in the circuit were all measured with the stadia, excepting those belonging to the base line, which necessarily formed a part of the closed circuit.

The distances and vertical angles were read twice, once as a foresight and once as a backsight, and the mean of both was used in plotting.

As each instrument point was occupied in running the circuit, readings were taken all around the circle, on all objects of topography, natural or artificial, which could have any bearing upon the proposed plans and estimates.

A sufficient number of readings was taken to permit the development of contours having a vertical interval of 2 ft.

All buildings, roads, railoads, property lines, ditches, canals, streams, high-water marks, orchards, swamps and rock outcrops were located, and notes were made of the character of the soil and substrata where exposed.

Sketches were carefully made in the field books, and were found to be of great service in plotting. Each stadia reading was numbered, beginning each morning with number one, and the locations of the principal readings, with their numbers, were shown on the sketches.

Each party was supplied in advance with a list of data for use in the locality where it was at work, which included the number, elevation, latitude and departure of all base-line instrument points, the true azimuth of the base-line courses and the approximate boundary of the territory to be surveyed. Mr. Himes.

With this information, it was an easy matter to find a starting or a closing point for a circuit, and to check the circuit after it was run.

In order to avoid complications in the adjustment of circuits for plotting, they were made as simple as possible. Along the base line, their length was limited to 2 miles. A secondary circuit was allowed to start from and return to a primary circuit, but no circuit was allowed to start from one circuit and close on another, or start from a circuit and close on a base line, save in exceptional cases.

Checking Survey.—In the beginning of the survey, the field work was checked by having each party plot its own work roughly on protractor sheets. At that time there were only two stadia parties at work. They were located very near together, and the checking was fairly well done. It was found, however, that to do the work thoroughly would require the addition of a draftsman and a computer to each party. Even with this change it would have been difficult to keep the checking close up to date, because the book which was in the field during the day was always needed at night for reducing the readings and completing the notes, so that the draftsman either had to be a whole book behind in his work, or else the field party must work first in one book and then in another, either of which methods was very objectionable.

Working Maps.—After the survey had been in progress for a time, it was decided by the Board that since working maps were needed, they could as well be made by the field parties instead of the plotting of the protractor sheets, thus saving the labor on one set of maps and securing the necessary check on the survey and a complete set of working maps at the same time.

It was desirable to plot the base line on these working maps by latitudes and departures, and, as they must be figured, it became convenient to check the stadia circuits by figuring their latitudes and departures and comparing results with those of the base line at the closing station.

The method pursued thereafter was, therefore, to check the stadia circuits by latitudes and departures immediately when closed, to send a book, when completed, to the headquarters of the survey, where the numerical work contained therein was first checked and the notes then plotted and errors or omissions reported back to the field for correction. The work was so divided among the stadia parties that each party could remain long enough in one locality to fill several books, and it was very seldom that a party had to go back to a previous location to look up errors.

Mr. Himes. After the winter work on Oneida Lake was completed, the force was so large, numbering for several months nearly 100 men, and the work was so scattered, that it would have been impossible to maintain a thorough control of it if the mapping had not been centralized and reduced to a system as above described.

Each night, after the men had returned to their boarding places and eaten their supper, the notes taken during the day were reviewed, the stadia measurements, when necessary, were reduced to the horizontal and the differences of elevation computed. The latitudes and departures of the circuit courses were figured, and if a circuit had been completed, the latitude and departure of the closing station were compared with its latitude and departure as given in the base-line notes. By this means, when errors occurred, they were always discovered promptly and corrected before they could create any confusion in the office work.

Precision of Work.—In order to show the degree of precision which was attained with the stadia, all circuits throughout the whole work which were closed on their starting points have been reviewed and their closing errors computed. The results are shown in Table No. 2, from which it may be seen that the error can readily be kept below $\frac{1}{500}$.

TABLE No. 2.

Error.	No. of circuits.	Maximum length of circuit.	Minimum length of circuit.	Average length of circuit.
Exceeding $\frac{1}{300}$	2	4 240	3 162	3 701
Between $\frac{1}{300}$ and $\frac{1}{500}$...	4	26 842	1 486	18 076
Between $\frac{1}{500}$ and $\frac{1}{1000}$...	10	31 000	2 140	12 787
Between $\frac{1}{1000}$ and $\frac{1}{1500}$...	7	7 015	1 818	4 779
Between $\frac{1}{1500}$ and $\frac{1}{2000}$...	6	56 048	2 589	21 163
Between $\frac{1}{2000}$ and $\frac{1}{3000}$...	7	31 027	4 597	11 555
$\frac{1}{3303}$.............	1	......	.....	16 522
$\frac{1}{3574}$.............	1	......	.....	5 894
$\frac{1}{10653}$.............	1	......	.....	4 885
$\frac{1}{17104}$.............	1	......	.....	13 688
Average error $\frac{1}{1058}$.........	40	56 048	1 486	11 782

The error in elevation seemed to bear no relation to the length of the circuit. It seldom exceeded 1 ft., and was generally less than 0.5 ft.

Organization of Party.—A stadia party usually consisted of five men; an instrument-man, a recorder and three rodmen. Sometimes a laborer was added to a party, temporarily, for the purpose of cutting brush or rowing a boat.

The instrument-man was required to direct the work of the party and attend to its needs. He selected the boarding place, paid incidental expenses and rendered accounts for board, team hire and sundries to the Assistant Engineer. He also made weekly statements of

the amount of work done, and kept a journal in which were recorded all events which might have a bearing on the progress of the work, such as storms, high water, the transfer of men from one party to another, and the loss of tools. Mr. Himes.

In the field he either kept notes or ran the instrument and had the general direction of the operations of the party.

The recorder acted as an assistant to the instrument-man, recording notes, running the instrument or taking charge of the party, temporarily, when the instrument-man desired to make a reconnoissance, or for some other reason was separated from the party.

The work of the rodmen was to carry the stadia boards in the field, to select points where readings were to be taken and report to the instrument-man any information they might obtain which would be of value to the survey.

An effort was made to allow each man in the party to acquire as much experience as possible, and for that purpose the recorder and instrument-man exchanged work occasionally with the rodmen. While the progress of the party was hindered a little at such times, it is believed that by increasing the interest of the men in their work the final result was benefited.

Field Notes.—Each instrument-man was required to keep his notes so clearly that any stranger, not familiar with the work, could read them readily. For this purpose, each book contained an index and a brief synopsis of its contents, and all notes copied from other books were thoroughly cross-referenced. For instance, when a stadia circuit was started from a base-line instrument point, a reference was made in the notes to the book and page of the base-line notes where the station was recorded, and also to the place in the level notes where the elevation was found.

No erasures were allowed in the notes, and the date, the number of the instrument, and the names of the observer and note-keeper were always recorded.

After the computations had been made, references were made to the computation books where the figures were recorded.

It was found nearly impossible to keep the notes written up and the reductions made by evening work, so every stormy day was used to catch up, and, on rare occasions, the whole party remained inside for a day to work on the notes.

Practice developed many convenient details in doing the work, which aided much in the interpretation and plotting of the notes. Some of the features of the work which have been stated were not begun until the work had been in progress two or three months, as they were the outgrowth of experience.

Instruments.—The instruments used were the usual railroad transits with stadia wires. There were two made by Buff & Berger, two by

Mr. Himes. Seelig, and one by Heller & Brightly. In the latter instrument the wires were adjustable, in the others they were fixed, but in all cases the value of the interval was determined, and the rods graduated accordingly, no attempt being made to adjust the wires in the Heller & Brightly transit.

Reduction of Readings.—The rods were graduated and the reductions made according to the formula deduced by S. W. Robinson, M. Am. Soc. C. E., of Michigan University. The tables computed according to that formula by Alfred Noble, M. Am. Soc. C. E., and Mr. Wm. T. Casgrain were used at first in making the reductions. Later, charts for doing the work were furnished by the Board, and their use greatly lessened the amount of necessary labor.

Graduation of Rods.—The style of graduation used on the rods was found to be especially adapted to the work, and is, therefore, exhibited in Fig. 4. The features of especial merit are, its simplicity and absence of fine detail, which avoids confusion, and the use of sharply-defined angles for divisions, which permits very precise readings.

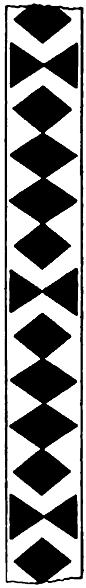

Fig. 4.

The rods were 4 ins. wide and 12 ft. long. The 5 and 10-ft. marks were painted red. Sights 2 000 ft. long were read very frequently on instrument points as well as for side readings.

Seasons.—The work was begun October 23d, 1897, and prosecuted continuously by one or more parties until its completion, November 5th, 1898. The work has been done in all seasons, in all kinds of weather, and in many kinds of country, including the wharves, buildings, bridges and canals in Oswego, the railroad yard in Utica, the irregular and thickly wooded hills of Sand Ridge and the open, level plains of the Mohawk Valley.

As to the comparative merits of the seasons for taking the topography, it was found that July and August were the most unfavorable months, and that the best time is in the early spring or late fall, when the weather is cool, the trees are bare and there are no crops or heat waves to interfere with the sight or impede the progress. In warm weather the atmosphere is too unsteady to permit long sights, the foliage is thick, and obstructs the view, and the summer heat renders the traveling more fatiguing, so that the rodmen cannot cover as much ground.

The progress of the party is frequently dependent upon the speed with which the notes are recorded, and in extreme cold weather the progress is lessened somewhat by the difficulty of using a pencil with cold fingers. In other ways, no especial difficulty was experienced in the winter time. During the winter of 1897–98, the snow caused no

more interference with the work than did the rain during the following summer. Mr. Himes.

The average number of days lost each month by the several parties, on account of storms, is as follows:

1897, October, 0.0; November, 3.5; December, 4.5.
1898, January, 1.0; February, 4.4; March, 1.4; April, 2.8; May, 4.7; June, 1.2; July, 0.7; August, 3.2; September, 1.5: October, 3.6.

Cost.—The cost of the stadia work per square mile, using the same items given by Mr. Landreth, viz.: salaries, maintenance, traveling expenses and supplies, was as follows:

Field work	$179
Mapping	101
Total	$280.

There have been excluded from the area several swamps which were surrounded with stadia work, but not penetrated. Their aggregate area is 13.4 sq. miles.

The average number of readings per square mile was about 1 440, or 2¼ per acre.

The minimum average area covered per day by one party on a single piece of work was 0.058 sq. mile. The maximum was 0.257 sq. mile. The average area covered per day by one party for the whole survey was 0.123 sq. mile.

In computing these averages, holidays and other lost time has not been deducted from the total number of days in the field.

About 83% of the area was mapped on a scale of 1:5 000. The remainder was mapped on a scale of 1:2 500.

The cost of mapping includes plotting soundings, test borings and triangulations. There were upwards of 12 sq. miles of soundings, and the cost of plotting these cannot be separated from the rest of the mapping. It would, perhaps, be more accurate to call the total area of mapping 133 sq. miles. The cost per square mile would then be $92. A vertical interval of 2 ft. was used in plotting the contours.

All elevations, determined either by sounding or with the stadia, were inked on the maps. This, it is thought, was not done in Mr. Landreth's work.

Conventions furnished by the Board were followed throughout the work, but while they were used sufficiently to show clearly the topography of the country, care was exercised to avoid everything in that line not wholly necessary, to the end that the maps might be a useful medium for an engineering study rather than an exhibition of artistic skill.

Mr. Himes. In making use of the foregoing data, it would be well to note that the maximum rate of progress was about four times the minimum. Also, in Mr. Landreth's data, the field work on Fish Creek for 5-ft. contours cost less than that for 10-ft. contours on Salmon River, while, on the Black River, the cost for 10-ft. contours was one-fourth of that on Salmon River.

It is surprising that on Fish Creek, where 620 readings per square mile were taken, the cost should be nine-elevenths of that on Salmon River, where 256 readings per square mile were taken.

On the Black River, Mr. Landreth took 131 readings per square mile, which is about one reading for every 5 acres. Eighty-five sq. miles were covered in that way. This small number of readings and the low rate of cost make it interesting to consider the cost of the United States Geological Maps, and to inquire whether work done in that manner would have served the purpose. In the New York State Engineer's Report for 1895, it is stated that the total cost of the United States Geological Maps in that State, prior to 1895, is at the rate of $10.62 per square mile.

It is noted in Table No. 1, that a large percentage of the work was taken directly from base-line stations. That practice was forbidden on the main surveys, it being considered that greater chances for error were afforded than where the topography was all taken from stadia circuits.

Considering the rapid movement of Mr. Landreth's party, it was undoubtedly better for him to use a large party and do the plotting at once.

The method of plotting outlined above was the outgrowth of conditions which had to be met, and should never be adopted without due consideration.

The results of the work appear to warrant a more general use of the stadia on preliminary surveys.

Mr. MacGregor. R. A. MacGregor, Assoc. M. Am. Soc. C. E.—In the survey of Baltimore, Md., the stadia method was used on more than 33 square miles. The maps were plotted to a scale of 200 ft. to 1 in., and all fences, roads, houses (with some details of the houses) contours at 5-ft. intervals, wooded areas, creeks, pasture and cultivated areas, etc., were shown.

The method used was somewhat similar to that described by Mr. Himes. The instruments were provided with fixed stadia wires; the horizontal and vertical circles were divided to read to 30 seconds, and the vernier of the vertical circle carried a level tube, each division of which had a value of 30 seconds.

The stadia boards were T-shaped in section and the figures were similar to those used on the United States Lake Survey. The meter was used as the unit of length on account of the convenience in using

the tables for difference of elevation, which were t
Messrs. Ockerson and Teeple of the United States
which give the elevation in feet for the distance in 1

Everything was plotted in the field, the map bei in pencil before leaving the locality. The shots w duced by the recorder, and plotted by the draftsman him. The topographer in charge then did the sketc

Each sheet contained ¼ sq. mile of territory, and and sometimes a great many more, points plotted the sent into the field. The co-ordinates of points and tances between the points were given, and closed cir tween them. The circuits were computed in the f greater than 1 m. was allowed, this being about the in plotting to 200 ft. to 1 in.

Elevations were read and plotted to 0.1 ft., a usually checked within 0.1 ft. On a run of 4 370 only 0.18 ft. between points determined by spirit leve

The average error of closure was about 1 in 700 b termined by precise traverse methods in which th 10 000. On a traverse of 1 000 to 1 200 m. the error w in 1 200. These ratios are all by actual calculation.

The cost of the 33.3 sq. miles was about $850 per sq stadia field work alone. The average number of shot about 10. The work done by the speaker, about 3 50(18 acres per day. The maximum number of acres s was about 75, with 207 shots, the minimum a small fra as the character of the territory varied considerabl small clusters of houses on most of the sheets, but on most thickly settled portion, there were 840 houses The number of shots per day averaged 180, with a max being plotted and the sketching done.

General H. T. Douglas, M. Am. Soc. C. E., was Ch the survey, and Mr. O. W. Connet was in charge of th division.

KENNETH ALLEN, M. Am. Soc. C. E. (by letter).—Tl graphical surveys is of little significance without full kı character of the ground, the extent of the survey, and expenses as transportation and board, which depend on

Of the remaining factors, the scale of the map, w controls the limit of error, is of prime importance. S is claimed that a certain survey was well conducted beca per unit of area than another, it means absolutely noth reason, such details as are given by the author are of val one to estimate the cost of similar surveys under similar

In connection with the recent investigations of

Mr Allen. Sewerage Commission, various topographical surveys were conducted by the writer from which the following five are selected, data regarding the cost of which are given in Table No. 3, and are, perhaps, of sufficient value to place on record.

The ground being similar in Surveys I and II, a comparison of their costs is of interest in showing the economy of reading compass bearings and setting up on alternate points, where this is sufficient, and practicable.

A comparison of the cost of Survey I with the costs of Surveys IV and V shows the increased expense due to plotting on a larger scale and halving the contour interval, the territories being fairly open in each case, and I and V having but gentle slopes. It will be noted, too, that IV and V are quite limited in extent in comparison with the others, which in part accounts for their greater cost per square mile.

TABLE No. 3.

Survey	I.	II.	III.	IV.	V.
Contour interval, feet	5	5	5	2½	2½
Scale of map	300′ = 1″	300′ = 1″	400′ = 1″	200′ = 1″	200′ = 1″
Actual working days	13	21	61	16	14
Total area, square miles	2.04	2.75	4.83	0.823	0.733
Area per day, square miles	0.157	0.131	0.079	0.0515	0.0524
Salaries per square mile	$54.90	$78.00	$1 40.20	$323.61	$256.21
Expenses per square mile	11.91	16.49	28.54	30.73	13.50
Cost per square mile	$66.81	$94.49	$168.74	$354.34	$269.71
Estimated cost per square mile, including superintendence	....	....	186.57	363.18	284.01

All these surveys were made by stadia and vertical arc measurements, running in circuits between established points, and referred to benches established by the wye level. The bearings were determined by the compass, except in Survey II, where the azimuths were turned off by vernier. The times given are for days worked only, and do not include time lost; while the costs given are for work in the field only, and do not include the map work done in the office. They do include, however, all expense for field work, including the plotting of the map on field sheets which were turned in to the main office and transferred by the draftsman to a sheet on which the lines or points of control had been plotted.

Surveys III, IV and V were made with considerable care and detail, being for the purpose of laying out filtration beds or plants for the treatment of the city's sewage.

Of Survey III, 47% was in timber, and 3% in marsh and water.

Of Survey IV, 12% was in water surface.

Of Survey V, 27% was in timber, 12% was in open marsh, and 6% was in water surface. Mr. Allen.

In Survey III the range of elevation was 125 ft. Some of the ground was quite rough and filled with a dense thicket of briar and underbrush. In the other areas the range of elevation did not, generally, exceed 40 ft.

The greatest error in closing the circuits of Survey III on the map, averaging about 1½ miles in length, was 80 ft., occurring in an unimportant place; the next was 30 ft., and the average 10 or 15 ft.

Two steel tape lines, which had been run through this area, were utilized as bases; while in Survey V, steel tape base lines were run from known positions determined by the United States Coast and Geodetic Survey.

The writer is interested in noting the use of a megaphone in the field. The means of communication between the rodman and the man at the instrument should be effective at the maximum distance and not liable to misinterpretation. A simple code of signals is believed to be preferable to verbal orders for directing the ordinary movements of the rodman, and for this purpose a whistle or small flag may serve; but there are times when some means of conveying fuller directions than can be done in this way is very desirable, and the megaphone may answer the purpose. Sight signals have the advantage of being effective where there are disturbing noises from winds, factories or waterfalls, while signals by sound may be preferable where there is more or less obstruction to the sight.

Wager Fisher, Jun. Am. Soc. C. E. (by letter).—The cost of the field work for topography with 10-ft. contour intervals on two territories, which were, respectively, 65% and 25% wooded, is given by Mr. Landreth as $66 and $16.50 per square mile. These results present an opportunity of determining, in a measure that is at least suggestive, how much the cost of such topography varies with the percentage of woodland. Mr. Fisher.

The writer, having computed the cost of a similar survey made in the summer and fall of 1899 by the New York State College of Forestry* of their track in the Adirondacks, in Franklin County, N. Y., submits the results in order that with others they may form a basis of comparison.

The territory was practically all wooded, and presented the usual difficulties of the mountainous and hilly country of the Eastern States. The area covered was 32 sq. miles, and, in the effort to find the cheapest method, both rectangular and irregular systems of stadia transit lines were tried. There were 172 miles of stadia circuit. The survey was made between July 1st and December 1st, 1899, but not continuously.

* Doctor B. E. Fernow, Director. Field work supervised by Professor H. N. Ogden, Assoc. M. Am. Soc. C. E., and Mr. William E. Mott, both of Cornell University.

Mr. Fisher. Comparing the seasons, summer and fall, differences in foliage were largely counterbalanced by differences in weather. The cost of maintenance was about 41% of the total expense.

TABLE No. 4.

Survey.	Percentage of woodland.	Cost per square mile.
Black River	25	$16.50
Salmon River	65	66.00
New York State College of Forestry	100	85.00

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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PAPERS AND DISCUSSIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

ON THE FLOW OF WATER OVER DAMS.

Discussion.*

By Messrs. GEORGE Y. WISNER and G. S. WILLIAMS.

GEORGE Y. WISNER, M. Am. Soc. C. E. (by letter).—In the investigation for developing a project for a deep waterway from the Great Lakes to the Atlantic, which was commenced by the United States Board of Engineers on Deep Waterways in the fall of 1897, it became evident, from the start, that the existing data relative to the flow of water over dams were inadequate for the accurate determination of river discharge where the depth on the crest of the dam was much over 1.5 ft. Mr. Wisner.

The uncertainty as to the value of the coefficients of weir formulas which should be used for dams of different cross-sections, and for different depths on the crest, made it apparent that additional investigations would be necessary before satisfactory estimates could be made of the value of water-power rights which may be modified, or of the amount of slope walls and bank protection which would be needed between the limits of the high and low-water stages of the proposed waterway.

At first, it was thought that extended observations with modern current meters at several of the principal dams in question would be necessary, but since the coefficient of the weir formula varies greatly with the shape of the dam, and as there are but few of the dams on the

* This discussion (of the paper by George W. Rafter, M. Am. Soc. C. E., printed in the *Proceedings* for March, 1900) is printed in *Proceedings* in order that the views expressed may be brought before all members of the Society for further discussion. (See rules for publication, *Proceedings*, Vol. xxv, p. 71.)

Communications on this subject received prior to June 22d, 1900, will be printed in a later number of *Proceedings*, and subsequently the whole discussion will be published in *Transactions*.

Mr. Wisner. Oswego, Mohawk and Hudson Rivers, which have similar cross-sections, such a method would have been incomplete and unsatisfactory, and was not attempted.

In the fall of 1898 the experiments of H. Bazin, published in the *Annales des Ponts et Chaussées*, became available, and established the coefficients for a great variety of different-shaped weirs, but, unfortunately, for depths of less than 1.5 ft. on the crests.

In December, 1898, the writer entered into correspondence with Professor Gardner S. Williams, M. Am. Soc. C. E., engineer in charge of the Hydraulic Laboratory of Cornell University, relative to extending the Bazin experiments for some of the forms of dams on the Oswego, Mohawk and Hudson Rivers, but, owing to the lateness of the season, nothing was done until the following April, when Mr. Rafter was requested to have his assistants construct experimental weirs of forms similar to those of the principal power plants on the proposed deep waterway routes, and Professor Williams was requested to install the necessary measuring apparatus to determine the coefficients for the different forms of dams to be experimented on, and to take general charge of the observations.

The success of the experiments was largely due to Professor Williams' ability as an experimenter, and to the untiring interest which he manifested in the prosecution of the work.

In authorizing these experiments, the Board of Engineers on Deep Waterways insisted that the standard sharp-crested weir should be thoroughly calibrated for all depths on the crest for which used. This, however, the observers were unable to accomplish in a satisfactory manner, but it is to be hoped this will be done in time to embody the results in this discussion.

Referring to the mean coefficient curve of the standard weir shown on page 306, it will be noted that for heads of from 2 to 3 ft., the coefficient is a minimum. So far as the writer is aware, there is no good reason for this, and it is probable that the peculiar shape of the curve may be due to incorrect calibration of the standard weir, and to the effect of velocity of approach and to side walls of a narrow flume for which the correction made is, apparently, largely a matter of judgment.

An examination of the curves on pages 272 to 289, inclusive, shows the same peculiarity for a large percentage of the experiments, and a study of the data and results indicates that incorrect determination of the effect of velocity of approach at the experimental weir and unknown resistances of the side walls of a narrow flume are the principal causes. Comparing the curve on page 289 with that on page 306, there is apparently no reason why one should be a regular curve and the other not, and, as observations made subsequent to those discussed in the paper indicate that the former is correct, it is a fair presumption that additional observations will very likely modify the shape of the latter.

GARDNER S. WILLIAMS, M. AM. S
speaker great pleasure were he able to
for the work of the author. Unfortun
from the opinions expressed in the p
points. He is fully aware that any c
far as they relate to the execution of the
upon himself, and, in extenuation, has
which he has the honor to represent is s
be glad, as occasion comes, to point out
of others. It will welcome any crit
instructions.

The matter of formulas will be passed
that there are some other errors than th
specifically, but it is probable that they
the speaker happened to receive, a consi
copy of that portion of the paper in wh
he believes there were no errors; so that
original translation was correct, and the
made in copying, as the author has sug

The most essential point in the pape
issue, is as to the use of Bazin's coefficie
in fact the author's use of Bazin's formula
differs from the formulas of Francis, I
Smith, in that it provides for the effect
coefficient; whereas all the others require
be corrected for the velocity of app
coefficient 3.33, is to be applied to th
rected. The Bazin coefficient is to be
observed. Therefore, the statement, or
the coefficients in the table on page 233 a
would be only true in the case of a weir of
speaking solely of weirs with end contrac
pressed weir is the only one that Bazin used
Cornell experiments. It is the only one c

The statement is also made that by ref
seen that there is very little change in the
weir of 6½ ft. is reached. That also needs
because the coefficient changes as the ve
and while the statement is true with the
to 1.6 ft., included in the table, it is not
for example, with a head of 100 cm., tha
11 ft. high, the discharge is about 0.6 o
weir 13 ft. high, according to Bazin's fo

* { *Annales des Ponts et Chaussées*, October, 1888
{ *Proceedings*. Engineers' Club, Philadelphia, 1

Mr. Williams. Of course, it is fully realized that in speaking of a matter of 0.6 of 1% in a weir measurement it is getting down rather fine; but we are to consider that accuracy in the measurement of water for power purposes or for consumption and accuracy in the design of a crest to discharge the flood volumes of a stream, are two different things. In the latter case, if one comes within 5%, he is doing very well. In the former, one should get down as near to 1% as possible.

The author, apparently, makes a misstatement on page 296, where he says that u—which by him is elsewhere called n, and was originally designated by the Greek letter μ by Bazin—is a coefficient which depends upon the height of the weir. Now, that is less than half the truth. It depends upon the height of the weir and the head over the weir, and it is given quite accurately by the formula $\mu = 0.405 + \frac{0.00984}{n}$ for English units. That is, it is given satisfactorily by a formula which does not involve the height of the weir at all. In other words, in speaking generally, the coefficient μ varies with the head over the weir and not with its height. The coefficient m varies with the head over the weir and the height of the weir, and m is the factor which enters finally into Bazin's formula, the formula being virtually made up of three formulas, first a formula for μ, then one for m, which involves μ, and then one for Q which involves m.

In the reduction of Experiments Nos. 20 and 21 on pages 298 and 299, if the speaker understands the author's use of the English language correctly, a curve has been plotted for the discharge of the upper weir by Bazin's formula, as given on page 296, which contains the coefficient providing for velocity of approach. From this the author has taken a value for the discharge. He has then from that discharge computed the velocity of approach, added to the observed head the correction for the head due to this velocity of approach, taken the value of Q for this increased head from the Bazin curve again, and he says that generally two applications of the process were sufficient. In the speaker's judgment, inasmuch as each application was adding a velocity head that did not belong there, it seems that two applications should have been sufficient. If the author desired to determine the discharge for a weir under this head with no velocity of approach, the correction should have been subtracted, not added.

It is possible that the speaker has misunderstood the author's method of reduction, but he has taken pains to refer these statements to several others, conversant with Bazin's formula and with hydraulics in general, and it has been generally agreed that the language is misleading, if the process has not actually been so. That is, the language simply means that instead of using the observed head H which should have been used with Bazin's formula, the head $H + h_a$, as in the Francis formula, was used, the result being that the computed discharge of

Mr. Williams.

the standard weir by Bazin's formula is thereby made too large. It may be pertinent to enquire why, if the author deemed a correction for velocity of approach necessary at the upper weir, he did not also apply one at the lower weir where the velocity was several times as great. Now, of course, granting that this mistake has been made, it can possibly be excused on the plea of a lack of familiarity with the French language, for it is often rather difficult to convert a foreign language into English and be sure that every point is brought out correctly, but it is difficult to excuse the statement on page 299 that: "Messrs. Fteley and Stearns have pointed out that for standard sharp-crested weirs the head should be measured about 6 ft. back from the crest," when, by turning to the paper by Messrs. Fteley and Stearns,* the following statement will be found:

"The head, if measured outside of the angle of pressure, should be taken far enough up stream from the weir to represent the height of the water surface above the beginning of the surface curvature, *i. e.*, at a distance from the weir equal to 2½ times its height above the bottom of the channel."

Our weir was 13 ft. high.

This brings us quite properly to the subject of weir experiments in general, and in the discussion of any hydraulic problem it is well to go back to the beginning and find out how much we really know about the thing in hand. We are dealing with weirs with end contractions suppressed, and so far as experiments have gone upon weirs of a sufficient size to be compared with those which are discussed in this paper, in which the discharge over such weirs has been measured volumetrically, the entire series of experiments is embraced in three investigations shown in Table No. 5.

TABLE No. 5.—Experiments upon Weirs with End Contractions Suppressed, in which the Discharge was Measured Volumetrically.

Observer.	Number of experiments.	Length of weir, in feet.	Height of weir, in feet.	Range of head, in feet.
James B. Francis....	17	9.995	4.60	0.78620 to 1.0600
Fteley and Stearns...	30	4.999	3.56	0.0746 " 0.8198
" " ...	10	18.996	6.55	0.4685 " 1.6038
Henry Bazin..........	67	6.562	3.72	0.194 " 1.012
" "	38	3.281	3.72	0.188 " 1.338
" "	48	1.640	3.296	0.191 " 1.779

It will be seen that Bazin's first series included nearly as many experiments as those of all the other investigators, and that, altogether, he has given us three times as many determinations of the flow over suppressed weirs volumetrically as the others have.

* *Transactions*, Am. Soc. C. E., Vol. xii, p. 47.

Mr. Williams. There is an important distinction between the methods of measuring head in Bazin's experiments and in the experiments of the American investigators. The latter adopted a position for reading the head 6 ft. up stream from the crest of the weir and about its level. Bazin read it 16.3 ft. up stream and at the bottom of the channel. The American experimenters took the water through a small opening in the side, in no case more than ¾ sq. in. in area, which communicated with a pail in which the surface was read by a hook gauge. Bazin used an opening 4 ins. in diameter which communicated to a chamber built alongside of his canal in which the head was read by a hook gauge. Now, it will be realized at once that it is to be expected that the velocity of the water flowing toward the weir would be greater at the American position than it would be 10 ft. further up stream, and, as any increase of velocity head or of velocity means a corresponding decrease of pressure head, it may be expected that for the same observation, if the head were measured at the American position, it would appear to be lower for the same discharge than if it were measured at Bazin's position. Therefore, for a given head, we should expect that Bazin would show a less discharge than would the American investigators.

Some may be inclined to doubt the importance of the variation in position in reading heads. Upon that point it may be said that in some investigations carried on last summer, the head was read directly at the crest of the weir by means of a tube set in the weir itself and communicating with the crest by small openings 6 ins. apart. These openings were about ¼ in. in diameter, and were bored vertically at the exact crest of the weir, which was the section adopted by the United States Board of Engineers on Deep Waterways for the proposed regulating weir on Lake Erie, *i. e.*, No. 19 of the author's series. At the time that these heads were read a tape was tacked upon the wall of the canal vertically at the crest, so that the top of the sheet could be read thereon at the same time that the pressure in the piezometer along the crest of the weir was read. With a head up stream of 97 cm. the tape read 70 cm. and the piezometer at the crest read 38 cm. There is the effect of velocity upon the head. The piezometer set on the crest of the weir read hardly more than half the depth of water which was actually flowing over the weir at that time, and as there were, altogether, somewhat in the neighborhood of thirty or forty experiments involving the crest piezometer, it may be affirmed that this was not an erratic observation.

It may be said further, that, since this condition exists, it is possible to use such a form of weir as a Venturi meter, particularly when submerged, and there is no doubt that a series of coefficients for a weir of the form of the United States Deep Waterways Section might be given to such a meter and would compare quite favorably, in accuracy,

at least, with the coefficients given by the
lar weirs. Time does not now suffice to g
It may be said, however, that as the cre
between the reading of the piezometer an
do not become equal up to 3-ft. heads, no
meter at the crest become equal to the
submergences of 4 ft. Now, as said abo
expected to give the higher head for a g
charge at a given head than those of the
when Bazin's formula is applied to the
investigators, in which the discharge was
a higher discharge than was observed.
are applied to the American experiments
they are applied to Bazin's experiments
than was observed. When we apply Bazi
ments, it fits most excellently.

TABLE No. 6.—COMPARISON OF OBSE
SUPPRESSED W

No.	Observer.	Weir. Length. Ft.	Height. Ft.	Observati Head. Ft.	Discharge.
1...	J. B. Francis.........	9.995	4.60	0.9760	32
2...	Fteley & Stearns.....	18.996	6.55	1.4546	112.
3...	" "	18.996	6.55	0.4685	20.
4...	" "	4.999	3.56	0.8118	12.
5...	" "	4.999	3.56	0.4569	5.
6...	H. Bazin.............	6.562	3.72	0.9794	21.
7...	"	6.562	3.72	0.5644	9.
8...	"	1.640	3.296	1.0158	5.
9...	"	1.640	3.296	0.5332	2.

2 Excess by Bazin's formula over Fteley & Stearns
1 " " " " " Francis
4 " " " " " Fteley & Stearns
3 " " " " " " "
5 " " " " " " "
8 " " " measurement over Francis' for
6 " " " " " "
7 " " " " " "
9 " " " " " "

Now, what is the meaning of this?
kind of water from that which the Am
one or the other has done the better wo
at once: Which? Bearing upon this poi
the best that has occurred to the speaker

Mr. Williams. the experiments of the several investigators were homogeneous in themselves; that is, whether they would coincide or whether they would show erratic variations from one side to the other of some mean. The criterion was to take the measured Q's and from them to derive an $n^{\frac{2}{3}} h$, which, when plotted as an abscissa with the observed head as an ordinate, would give a straight line if n were constant. Of course, since n is not constant, but increases with the head at the higher heads, it does not give a straight line, but the variation is not great for the range of the experiments. That criterion showed clearly that the results of the experiments were homogeneous in themselves, and indicated a high degree of relative accuracy. That is, if one was right the other was right in the same series. Pains were then taken to study particularly the arrangement by which the quantity was measured, and the conclusion has been that the devices used by Mr. Francis and by Messrs. Fteley and Stearns for starting and stopping the flow and also for determining the height of water in the measuring basin were more delicate, and capable of more accurate work than were those of Bazin, so that, patriotism aside, it seems that greater confidence may be reposed in the observations of Mr. Francis and of Messrs. Fteley and Stearns than in those of Mr. Bazin, although the latter has made three times as many as the others. As already stated, the Francis formula gives results below Bazin for the lower heads, but the discharge curves of the two formulas cross at a head of about 1.4 ft. on a weir 11 ft. high, and above that Bazin gives lower discharges. Comparing the Francis formula with the discharges observed by Fteley and Stearns at the higher heads there is some evidence that the Francis formula gives too low results with such conditions, and therefore it seems that Bazin's formula is probably on this account the less accurate at the high heads.

Whether or not all wish to agree with the deductions as to the effect of velocity of approach or velocity past the openings and as to the relative reliability of the work of the various investigators, they will probably agree that if Bazin's formula is to be used, the head should be measured as Bazin measured it, and if the Francis formula is to be used, the head should be measured as Francis measured it. But the question naturally arises, what difference does it make? A few days ago the speaker had the privilege of performing some experiments to see what difference it made whether the head was measured one way or another. The weir in question was a small decimal over 20 ft. in length. Its height was 5.85 ft. above the channel of approach. End contractions were suppressed. At a point 10.3 ft. up stream from the weir there was a pipe, 1 in. in internal diameter, set 1 ft. above the bottom of the channel of approach, transversely to the direction of flow. This pipe was perforated on its bottom with holes about $\frac{3}{16}$ in. in diameter every 3 ins. in its length. One end of this pipe was connected by means of a $\frac{3}{4}$-in. pipe and a $\frac{3}{4}$-in. hose, to a hook-gauge pail,

from the weir. At the other side of the channel c
similar recess. This transverse pipe was a device wh
used for measuring the head upon this weir, whic
incidentally, is a somewhat important one. As a res
gations at Cornell, the reliability of a measurement
was questioned; and, in order that there might be
important work which was in hand, the plate which
the recess on the opposite side of the canal was tap
point 6 ft. from the crest of the weir and 0.35 ft. below
pipe screwed in, the face of which was filed off flush w
channel. A wooden plug was then driven into the p
off flush; a ¼-in. hole, perpendicular to the side of
bored, thereby nearly reproducing the device used b
Messrs. Fteley and Stearns. The transverse pip
through the side of the canal to a second hook-gauge
chamber. The hook-gauge pail was removed, and in
nected one of two glass tubes of ¾-in. inside dian
mounted rigidly on a board in front of a common
2-millimeter divisions. The other tube was connecte
what will be designated as the Francis, piezomete
hook gauges were then clamped to the crest of the we
was raised to within about ¼ in. of the crest, and by
the crest with the portable hooks the reading of the
permanent hook gauge for the crest of the weir, wer
water level, so that there might be no mistake as to t
instruments.

A series of investigations was then made to de
the tube which was connected to the transverse pie
correspondence with the hook gauge, and it was foun
remarkable constancy for a wide range of head, the
that the tube showed continuously about 0.003 ft.
the hook. It was therefore considered that any
appeared in the readings of the two glass columns
difference in head by the two piezometers. Omittin
it was found that at low heads the transverse piezo
and that at high heads it read low, and with a he
the weir it made 2½% difference in the discharge whe
was taken by the Francis piezometer or by transverse
discharge by the Francis piezometer being 2½% gre
transverse piezometer. That will give an idea of the i
using any formula, of measuring the head as it was
the formula was devised. If the head is measured
the formulas may or may not apply. Unfortunately
provision was made in the construction of the plant f

Mr. Williams. head upon the standard weir by either of the recognized methods. It was, therefore, necessary to resort to some other means, and, without knowing positively what the result would be, the transverse piezometer, which has been described by the author, and which coincides quite closely with that at the weir, which has just been discussed, was adopted. There was some suspicion when it was put in that it might lead to trouble, and it was expected that such checks could be made on the work as it went along as to detect such an error at once if it should occur. But the great pressure which was brought to bear to hurry the experiments, and the other duties which were demanded of the investigators, prevented the working up of those experiments, even the first of them, until the series was nearly completed. Then it became apparent that there was something wrong with the piezometers, and, accordingly, there was set, alongside the one at the lower weir, another, which was flush with the bottom of the channel of approach, and which probably coincided quite closely with Bazin's opening in the side of the canal at the bottom, although, of course, it did not coincide exactly. It was then found that, at the highest heads used, the difference in head, as measured by the two piezometers, amounted to about 10 cm., or about 0.3 ft., which means considerable in the discharge of the weir. The author has given, on page 305, a correction curve which he applies in these experiments, and which is probably the best that could be done under the circumstances. The discovery of such an error at the lower weir led at once to the conclusion that there must be something the matter with the piezometer at the upper weir also, but time did not permit an investigation previous to the completion of the series of experiments described. As the speaker was, at the close of this investigation, requested by the Board of Engineers on Deep Waterways to continue the work by an investigation of the discharge over the "Deep Waterways" Section, so-called, Section 19, a rounded crest with a 45° up-stream slope, at various degrees of submergence, and also with free discharge, it gave an opportunity to make some further investigations upon the foregoing question, and in order to measure the head in another way at the standard weir three pipes were set longitudinally, that is, parallel to the direction of the flow of the water at a point 6 ft. above the bottom of the canal and 28 ft. up stream from the weir and with their down-stream ends projecting about 6 ins. down stream from the plane of the old up-stream transverse piezometer. These pipes were 6 ft. long, $\frac{3}{4}$ in. in diameter, nominally, and were perforated for 1 ft. at their down-stream end with holes on the quarter, so that there was around the pipe a ring of holes 1 in. apart for a distance of 1 ft. at the lower end. These pipes were connected together at the down-stream end, the up-stream end being plugged. A $\frac{3}{4}$-in. pipe connection, similar to those of the transverse piezometers, was carried through the bulkhead, so that they might be connected to one

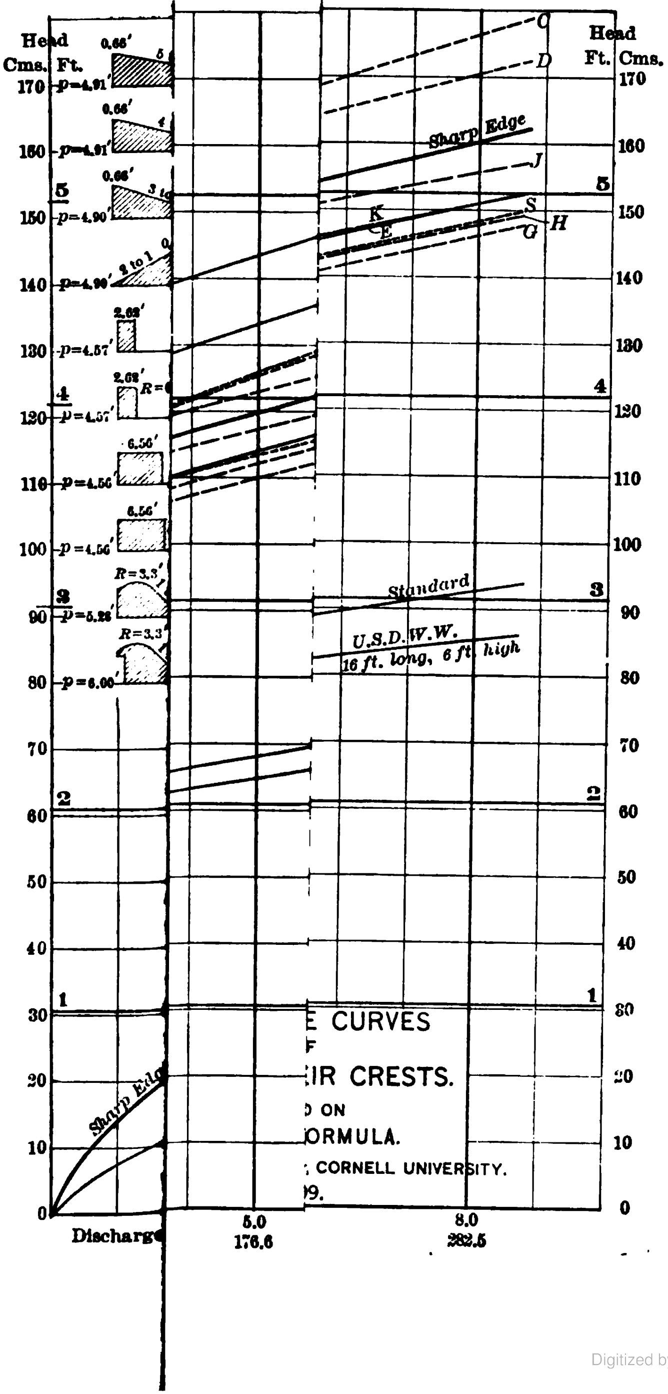
side of the tube gauge. The reason for adopting this type of piezometer was that Mr. FitzGerald* records that he investigated the head, when measured in pipes, by a device of this sort in which the perforated pipe is laid upon the bottom of the large pipe, in comparison with the head as given by a piezometer consisting of a chamber surrounding the pipe and communicating with it by holes in a plane at right angles to the axis of the pipe, and he found that there was apparently no difference. So that on the strength of those experiments it was ventured to assume that this would probably give a correct reading of head at the point where it was wished to measure it, *i. e.*, at a point corresponding to Bazin's position, if there is such a thing as a correct reading of head. About 40 experiments were made and they showed, at the highest heads observed, a little over 3 ft., that there was a difference of 3 cm. in the head as read by the transverse piezometer and as read by the longitudinal piezometer, the new longitudinal piezometer giving a head 3 cm. higher than did the old up-stream transverse piezometer. As soon as these data were obtained a memorandum of it, sufficient to locate a correction curve, was furnished to the author, who, however, decided to reject the readings of the up-stream piezometer and to adopt those of the middle one, which was 10 ft. back from the weir. Now, having simultaneous observations on the middle piezometer and the up-stream piezometer, and having a series comparing the new piezometer with the up-stream piezometer taken through the later investigation, the new piezometer readings, assumed to be correct, were plotted as a straight line at an angle of 45° with the axes on which the heads were laid off, and with this the old transverse up-stream piezometer and the middle piezometer were plotted. The differences could not be detected at a head of 5 cm., $\frac{1}{6}$ ft., but at 3.3 ft. the difference between the new and the old upper piezometer was 3 cm., the latter being low at all points. The curve of the middle piezometer started above the longitudinal one and reached a maximum difference at a head of about 2½ ft., where it was about 1 cm. high, and then dropped rapidly, appearing to cross the longitudinal piezometer at about 3.3 ft., after which it would read low. Now, of course, it cannot be affirmed that the new piezometer is a correct one to use with Bazin's formula, but, in view of the experiments of Mr. FitzGerald on the Rosemary Syphon, and in view of the comparison by the tube gauge of the transverse piezometer and the Francis piezometer it seems safe to infer that the new piezometer is the best one to use for the heads on this weir, or is as nearly correct as anything that we have. Furthermore, it was discovered that the head, as given by the new longitudinal piezometer, was much more steady and there was less vibration, less change and less pulsation than in the head as observed by the other. In order to complete the Mr. Williams.

* "Flow of Water in a 48-in. Pipe," *Transactions*, Am. Soc. C. E., Vol. xxxv, p. 259.

Mr. Williams. whole subject, an attempt was made to determine whether the manner of opening the gates, or the way in which the water was admitted to the chamber, would have any effect on the correction, and to that end all the head gates were opened a short distance, allowing the water to enter at the bottom of the chamber and flow over the weir. Then, at times, only two gates were opened, and they were opened wide, so that the water would enter from the bottom clear up to the middle of the height of the weir, but, with the maximum variation of flow which it was possible to obtain, there was no appreciable difference in the reading of the two piezometers, so that it is assumed that the corrections which were determined are probably as reliable as could be determined with the apparatus used, in which the heads were read in divisions of about 1 mm. Now, applying these corrections to Experiments Nos. 20 and 21, and applying Bazin's formula without the correction for velocity of approach, it appears that for heads on the standard weir running up to 23 cm., which means a head on the lower weir of about 1½ ft., that is, so long as the head on the lower weir was within the range of Bazin's investigations, the difference in the discharge, as given by the two weirs, is less than 2 per cent. In fact, for heads on the lower weir from a little less than 1 ft. up to about 1⅓ ft., the difference is less than 1 per cent. But, as soon as the head gets above that point the discharges depart very rapidly, the lower weir showing the higher discharge; and the variation ranges from 3% up to nearly 6 per cent.

TABLE No. 7.—COMPARISON OF SIMULTANEOUS DISCHARGE OVER UPPER AND LOWER STANDARD WEIRS BY BAZIN'S FORMULA.

Experiment No.	UPPER WEIR. 16 ft. long, 13.13 ft. high.			LOWER WEIR. 6.56 ft. long, 5.2 ft. high.		Percentage of excess of lower over upper weir.
	Obs. H. Transverse Piezometer.	Cor. H Longitudinal Piezometer.	Q = cubic meters per second.	Obs. H. Flush Piezometer.	Q = cubic meters per second.	
	Cm.	Cm.		Cm.		
13.........	12.075	12.28	0.3998	22.744	0.4058	+1.378
12.........	15.023	15.30	0.5499	27.855	0.5484	−0.278
6.........	18.069	18.39	0.7178	33.175	0.7124	−0.203
11.........	21.294	21.65	0.9133	39.419	0.9214	+0.886
7.........	23.759	24.16	1.0720	44.000	1.0915	+1.820
10.........	26.810	27.21	1.2785	49.699	1.3305	+4.067
1.........	29.658	30.16	1.4900	55.213	1.5456	+3.732
8.........	29.723	30.22	1.4945	55.128	1.5436	+3.283
9.........	37.437	37.90	2.0805	68.238	2.1440	+3.052
5.........	43.812	44.22	2.6280	80.566	2.7772	+5.677
4.........	58.110	59.00	4.4800	105.639	4.2645	−4.810
3.........	72.710	74.22	5.7300	130.286	5.9582	+3.981
2.........	79.565	81.69	6.6200	142.557	6.8881	+4.041


Head Cms. Ft.
Head Ft. Cms.
Sharp Edge
Standard
U.S.D.W.W. 16 ft. long, 6 ft. high
E CURVES
F
IR CRESTS.
D ON
ORMULA.
CORNELL UNIVERSITY.
Discharge
5.0
176.6
8.0
282.5

Mr. Williams.

In view of the conditions existing during the experiments it does not seem possible that the flow into the canal between the two weirs could have exceeded the leakage from it at the lower gates. In other words, it appears clearly impossible that more water passed over the lower than over the upper weir on this account; and yet, according to the observations and formula, there is an excess of about 4% in the discharge of the lower weir, to be accounted for at the high heads. It is to be noted, however, that the observations at high heads were taken in general from high to low, and as the gauges were only read to 2-mm. divisions, it is quite possible that the canal surface may have been falling when it appeared to the observer to be stationary, and hence a somewhat greater discharge have passed the lower than the upper weir.

A portion of the excess possibly may be accounted for, that is, about 0.6 of 1%, by the narrowing of the notch of the lower weir due to the pressure of the water in the canal sides. It was contracted toward its top slightly. That was not measured accurately. But such measurements as were made went to show that about 2 ft. from the top of the weis it was contracted nearly ½ in. This would make a difference of about 0.6 of 1% in the discharge at the higher heads. But that falls far short of accounting for all the difference. Mr. Wisner, in his discussion, has suggested that the effect of the side walls may very properly be considered to have something to do with this condition, and the extreme roughness of the sheet as it passed over the lower weir at the high heads will probably explain what remains.

In a later investigation it happened that there was obtained, incidentally, some notion of the effect of such roughness. There happened a repetition of two experiments in which the experimental weir remained in the same condition, while on the standard weir there were additional baffles between the two experiments, so that in the second case the water approached the weir much more smoothly than in the former. It appeared that with such roughness as existed in the standard weir, with heads of about 1½ ft. before smoothing, there was about ½ cm. more head required to deliver the same quantity of water than with the smoother approach. That is to say, if the water approaches the weir with high commotion a higher head will be required to discharge a given quantity than when the approach is smooth. The commotion at the standard weir, in the later experiments, was not to be compared with the commotion at the lower weir in the case of the high heads. In the latter, and next to the wall, there was a roll, then came three crests and depressions, the bottom of the depressions being sometimes nearly 6 ins. below the crests. It seems, therefore, tha twhen a proper correction is made for the effects of the roughness, the two weirs would come quite closely together. Of course, this reduction which has been made is simply applying

Mr. Williams. TABLE No. 8.—EXPERIMENTS UPON VARIOUS WEIRS FOR UNITED STATES BOARD OF ENGINEERS ON DEEP WATERWAYS AT THE HYDRAULIC LABORATORY OF CORNELL UNIVERSITY.

(1) Weir and No. of experiment.	STANDARD WEIR, 16 FT. LONG, 13.13 FT. HIGH.			EXPERIMENTAL WEIR, 6.56 FT. LONG.		
	(2) Obs. H. Cm.	(3) Cor. H. Cm.	(4) Discharge. Cu. met.	(5) Obs. H. Cm.	(6) Cor. H. Cm.	(7) Description.
C = 3—1.	86.802	89.50	7.6080	158.006	170.00	5 to 1 up-stream slope, 8-in. flat crest; height of weir, 4.91 ft.
2.	59.205	60.10	4.1650	105.648	110.00	
3.	29.061	29.50	1.4420	51.841	52.42	
D = 4—1.	87.478	90.23	7.7040	155.159	166.45	4 to 1 up-stream slope, 8-in. flat crest; height of weir, 4.91 ft.
2.	72.707	74.16	5.7200	129.889	137.58	
3.	58.210	59.10	4.0580	103.987	108.02	
4.	43.714	44.15	2.6210	78.126	80.12	
5.	29.449	29.90	1.4715	52.260	52.85	
6.	14.583	14.81	0.5227	27.587	27.82	
E = 5—1.	87.146	89.80	7.6470	137.816	146.70	3 to 1 up-stream slope, 8-in. flat crest; height of weir, 4.90 ft.
2.	73.47	74.95	5.8155	117.268	123.00	
3.	57.78	58.59	4.0030	95.72	98.80	
4.	42.114	42.48	2.4675	74.04	75.72?	
5.	30.342	30.76	1.5340	50.02	50.65	
G = 7—1.	87.514	90.28	7.7080	134.399	142.75	2 to 1 up-stream and 2 to 1 down-stream slopes, 8-in. flat crest; height of weir, 4.895 ft.
2.	72.700	74.16	5.7200	114.005	119.20	
3.	57.998	58.84	4.082	93.813	96.72	
4.	43.778	44.19	2.6240	72.802	74.50	
H = 8—1.	87.168	89.82	7.6500	135.558	144.00	Same as G with $\frac{1}{4}$-in. mesh $\frac{1}{16}$ in. thick, wire-cloth netting on up-stream slope.
2.	71.970	73.40	5.6320	115.086	120.50	
3.	57.854	58.66	4.0120	94.270	97.27	
4.	43.958	44.36	2.6420	72.636	74.35	
5.	28.804	29.20	1.4210	49.217	49.77	
I = 9—1.	87.350	89.95	7.6660	138.198	147.10	2 to 1 up-stream and 5 to 1 down-stream slope, 4-in. flat crest; height of weir, 4.94 ft.
2.	72.307	73.70	5.6675	117.366	123.00	
3.	57.702	58.51	3.9975	96.392	99.62	
4.	44.050	44.45	2.6495	74.510	76.22	
5.	29.404	29.82	1.4660	50.418	51.00	
J = 10—1.	88.406	91.19	7.8220	143.95	153.82	Vertical faces, 2.62-ft. flat crest; height of weir, 4.57 ft.
2.	73.626	75.05	5.8260	125.895	132.94	
3.	59.154	60.05	4.1610	106.66	110.98	
4.	45.168	45.58	2.75[illegible]	85.57	87.99	
5.	30.090	30.48	1.5115	60.676	61.70	
K = 11—1.	88.412	91.22	7.8800	140.010	149.15	Same as J, with 4-in. radius quarter-round added to up-stream corner.
2.	74.006	75.61	5.8930	121.39	127.70	
3.	58.592	59.48	4.098[illegible]	101.751	105.60	
4.	44.270	44.69	2.6690	80.511	82.52	
5.	30.540	30.90	1.5485	57.895	58.80	
L = 12—1.	72.662	74.10	5.7150	144.640	154.55	Vertical faces, 6.56-ft. flat crest; height of weir, 4.56 ft.
2.	57.89	58.71	4.0180	120.580	126.80	
3.	43.816	44.23	2.6300	93.760	96.75	
4.	29.40	29.80	1.4645	64.938	66.30	
M = 13—1.	72.632	74.08	5.7100	136.098	144.70	Same as L, modified as K.
2.	58.23	59.09	4.0550	111.67	116.60	
3.	43.71	44.16	2.6220	86.019	88.52	
4.	29.396	29.80	1.4645	59.010	60.02	
5.	14.982	15.21	0.5445	30.585	30.80	
N = 14—1.	87.22	89.90	7.6600	146.831	157.05	Rexford Flats Model; height of weir, 4.53 ft.
2.	72.59	74.00	5.7010	124.720	131.60	
3.	58.018	58.90	4.0378	101.567	105.40	
4.	43.722	44.18	2.6230	78.314	80.25	
5.	29.156	29.57	1.4470	53.549	54.25	
6.	14.442	14.66	0.5150	27.812	28.05	

TABLE No. 8—(*Continued*).

Mr. Williams.

(1) Weir and No. of experiment.	STANDARD WEIR, 16 FT. LONG, 13.13 FT. HIGH.			EXPERIMENTAL WEIR, 6.56 FT. LONG.		
	(2) Obs. H. Cm.	(3) Cor. H. Cm.	(4) Discharge. Cu. Met.	(5) Obs. H. Cm.	(6) Cor. H. Cm.	(7) Description.
O = 15—1.	73.262	74.75	5.7910	122.953	129.55	Same as N, with rounded corner as in K and M.
2.	58.393	59.24	4.0745	101 191	105.00	
3.	43.555	44.00	2.6070	77.557	79.52	
4.	29.618	30.10	1.4848	53.041	53.65	
P = 16—1.	72.63	74.10	5.7120	12'.582	126.75	Little Falls Model, 3¼ to 1 up-stream slope; height of weir, 4.57 ft.
2.	58.31	59.18	4.0660	96.957	103.55	
3.	43.56	44.00	2.6075	76.240	78.02	
4.	29.05	29.50	1.4425	51.514	52.00	
Q = 17—1.	72.878	74.40	5.7485	121.04	127.30	Little Falls Model; 3 to 4 up-stream slope; height of weir, 4.57 ft.
2.	58.326	59.23	4.0725	99.763	103.32	
3.	44.29	44.73	2.6730	76.552	78.39	
4.	29.084	29.50	1.4420	51.019	51.57	
5.	18.004	18.35	0.7140	32.038	32 40	
R = 18—1.	22.900	23.30	1.0170		33.523	Indian Lake Model; height of weir, 4.65 ft.
2.	86.074	88.60	7.4835		149.362	
3.	71.992	73.40	5.6325	Correct head read on flush piezometer.	125.693	
4.	57.586	58.38	3.9810		100.766	
5.	43.314	43.75	2.5850		75.427	
S = 19—1.	14.338	14.59	0.5118		27.04	Submerged section, round crest, 1 to 1 up-stream slope; height of weir, 5.28 ft.
2.	28.435	28.85	1.3950		51.36	
3.	85.786	86.86	7.4580		142.123	
4.	70.915	72.36	5.5140		119.442	
5.	56.434	57.30	3.8710		97.858	
6.	43.812	44.38	2.6405		77.246	
7.	28.962	29.38	1.4380		53.42	

Bazin's formula to the two weirs and computing a discharge for weirs of that height according to his formula without any further corrections whatever. At the time these investigations were begun it was said by the Board of Engineers on Deep Waterways that if we could give them results within 6% of accuracy they would be abundantly satisfied. In the speaker's opinion, the results come within that range. He would not claim more. The Hydraulic Laboratory Staff has performed experiments since that time, in connection with the Croton Water-shed investigations, which come far nearer to accuracy than 6%; but, so far as those which are given in the paper are concerned, it is very questionable if they can be depended upon within less than 6 per cent. Now, it appears to the speaker—it may be a notion in which he is peculiar—that in presenting the results of an investigation of this kind to this Society, in putting the observations upon record, forever as it were, it is most proper to present them first as nearly as possible as they were taken, to keep quite distinct the data which are facts and the data which are conclusions, to present the experiments as they were made with as little reduction as possible, so that in the future the investigator may determine for himself, in the light of such new knowledge

Mr. Williams. as he may then have, just what reliability is to be put upon the observations, and what lessons are to be drawn from them.

The speaker would criticise the author for having presented here a paper in which practically all is reduction, and there is no getting behind his returns, whatever we may discover in the future as to the flow over weirs. So far as the data in this paper are concerned, there is little that we can go back to and make a rigid comparison with. That which has been presented is deduced from a computed discharge of the standard weir, which has been shown to be fundamentally in error. It then follows that the whole array of coefficients and coefficient curves on pages 272 to 289, inclusive, are similarly in error and therefore correspondingly reduced in value. This error probably ranges from zero to 3 per cent.

On Fig. 4 there is the following note: "The correction for Velocity Head $\left(\frac{v^2}{2g}\right)$ as used in Reducing the Experiments is in Effect Equivalent to $h_v = 1.33 \left(\frac{v^2}{2g}\right)$ for position of Piezometer 6.0 Ft. Back of Weir." Upon what authority this statement is made the speaker is unaware, but if there are any data upon which such a statement can be legitimately based it is to be regretted that the author did not give a reference thereto. So far as the Cornell experiments are concerned, there is nothing to lend support to such an assertion, and until some facts are brought to support it, it is only entitled to consideration as a rather positively expressed opinion, which, in the speaker's opinion, is contrary to fact.

In order that the results of this investigation may be properly on record, Table No. 8 is presented, in which Column 1 gives the number of the experiment in its proper series; Column 2 gives the observed head on the standard weir; Column 3, this head corrected to that read by the longitudinal piezometer 28 ft. up stream from the weir and 6 ft. above the bottom of the channel of approach; Column 4, the discharge per second, in cubic meters, by Bazin's formula,

$$Q = \left[0.405 + \frac{0.003}{h}\right] \left[1 + 0.55 \left(\frac{h}{p+h}\right)^2\right] l\, h \sqrt{2\, g\, h},$$

where p = the height of the weir = 4.002 m. = 13.13 ft., and l = length of crest = 4.8768 m. = 16 ft.; Column 5 shows the head simultaneously observed upon the experimental weir at the up-stream transverse piezometer; and Column 6 this head reduced to the flush piezometer, or the head observed upon the flush piezometer. All heads given are the means of those observed during the time of the experiment. These heads have been recomputed from the original field notes.

Series A, B and F (Nos. 1, 2 and 6 of the author), have been omitted, the results being too questionable to warrant insertion with the

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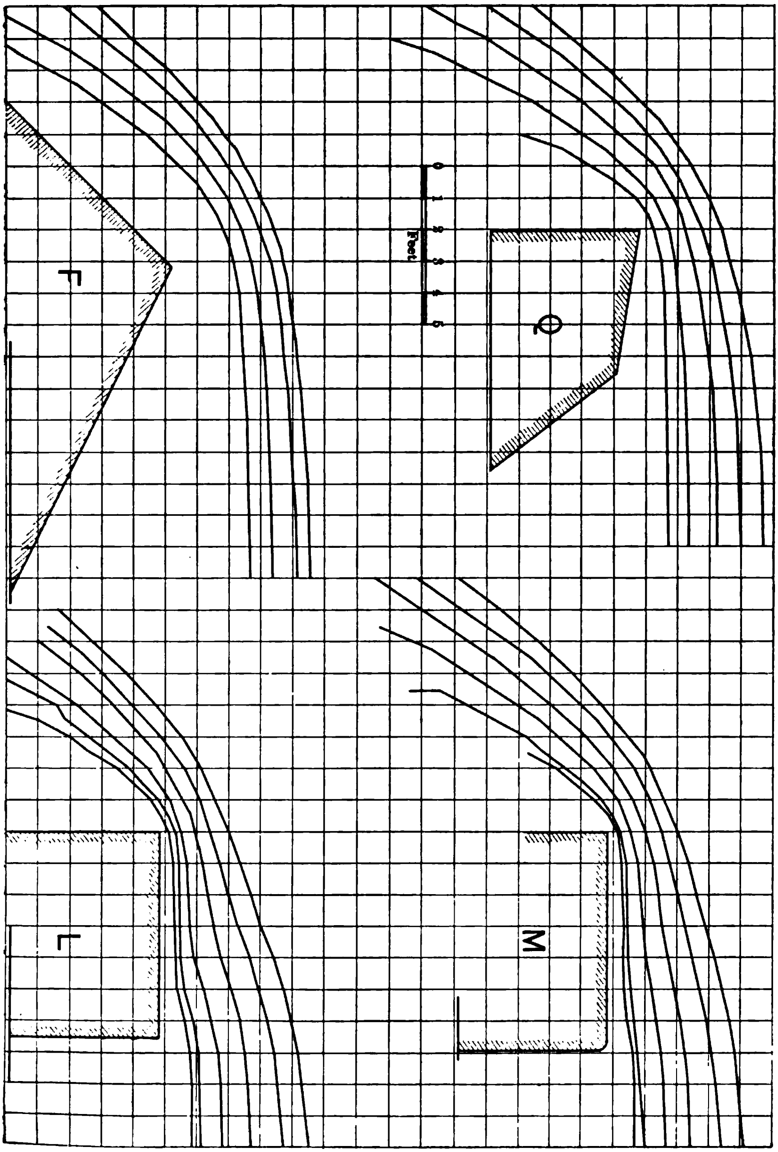

Fig. 21.

Mr. Williams. others, and E (author's No. 5) is considered as quite possibly inaccurate. The crests of the experimental weirs were approximately 2 m. = 6.56 ft. long.

While the absolute values determined by this investigation may be considerably astray, because of uncertainty, within at least 3% of the quantity of water passing the standard weir, the relative discharges of the several experimental weirs are of great interest, and on Plate XXV the discharge curves of several of the types are shown, these curves being based upon that of the 16-ft. standard weir computed by Bazin's formula. These weirs were all of approximately the same height, the range being from 4.6 to 5.3 ft., so that from the plate one may readily see the effects upon the discharge caused by crests of various forms.

It will be noted that the discharge curve of the experimental sharp-edged weir divides the upper group of curves about in halves, those weirs whose curves fall above it giving a less discharge for a given head than does the sharp-edged weir.

One very interesting point is the behavior of broad flat crests. As seen by L, they give, at the lower heads, much less discharge than the standard, but, as is shown by J, and already pointed out by Messrs. Fteley and Stearns, and by Bazin, when the head reaches a point at which the sheet jumps from the up-stream edge clear or nearly clear of the down-stream corner, and the space between the sheet and crest becomes filled with eddying water, the discharge is very notably increased; so much so in the case of J that it exceeds that of the sharp-edged weir at 4.5 ft. head. The curves M and K show the increase of discharge due to building on a 4-in. radius, quarter round, to the up-stream corner of L and J; this rounded edge adding over 11% to the discharge at a 4-ft. head in both cases.

The effects of long and short back or up-stream slopes are shown by the curves C, D, E, G and S. C being 5 to 1; D, 4 to 1; E, 3 to 1; G, 2 to 1, with a 2 to 1 down-stream slope added; and S, 1 to 1 with a $3\frac{1}{3}$-ft. radius round crest. As stated by Bazin, when the inclination of the up-stream face of the weir is with the top down stream, the tendency is to suppress the contraction of the sheet as it goes over the crest, and thereby increase the discharge, but if the up-stream slope be too gradual, the frictional resistances along it may be sufficient to counteract the gain in discharge from suppression of contraction. This appears to be the case with C and D, and, at low heads, with S, when the curved crest partakes of the nature of a long slope. At higher heads the 1 to 1 back-slope becomes effective and the discharge increases above that of the standard weir. From the upper curves, the weir G appears to have the maximum discharging capacity, but this seems to be in part due to the fact that the entry of air under the discharging sheet was restricted with it, but not with the others. The

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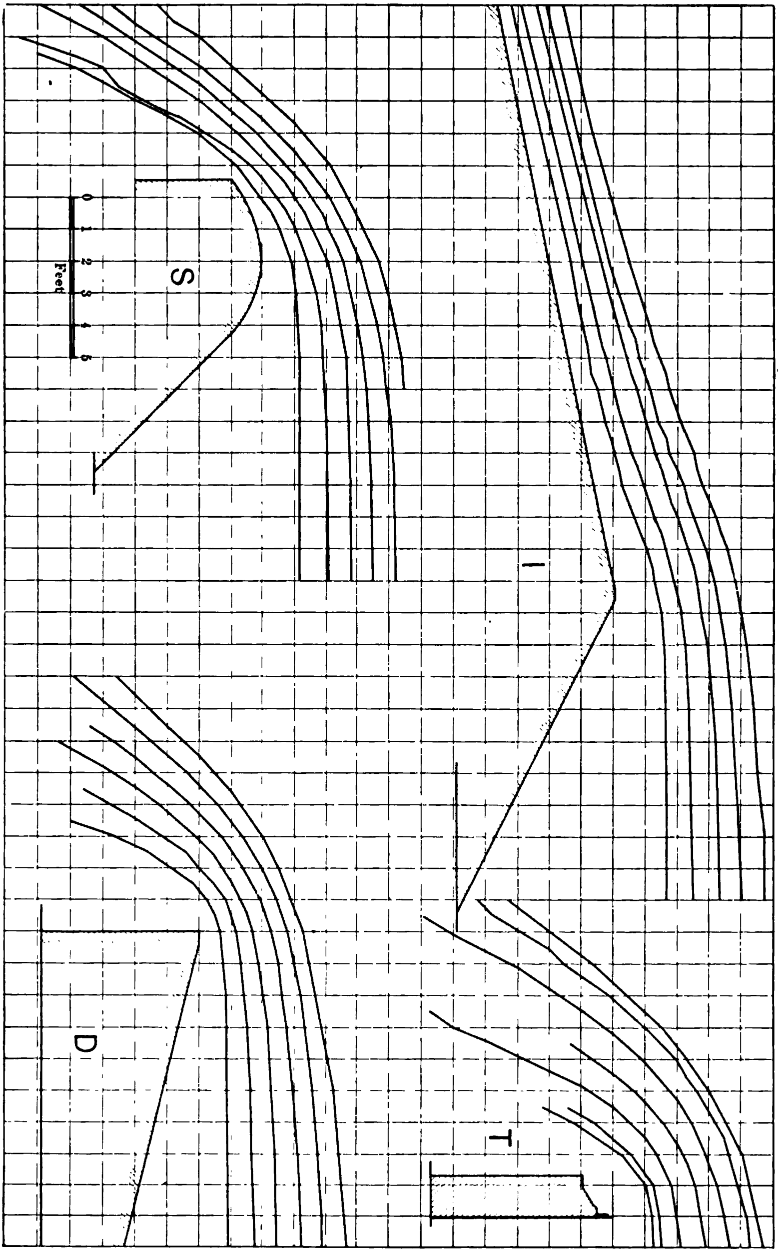

Fig. 22.

Mr. Williams. later experiments upon the United States Deep Waterways Section, 16 ft. long and shown with the 16-ft. standard weir in the two lower curves of the plate, wherein air was not admitted under the sheet, gives very nearly the same discharge at 3 ft. head, both giving over 11% more than the sharp edge. The difference between curves *G* and *H* shows the effect of adding wire-cloth to the up-stream face of the weir *G*. Some later experiments indicate that the difference of discharge between a crest of dressed pine and one as rough as ¼-in. mesh wire-cloth will hardly amount to 3 per cent.

During this investigation, at the suggestion of Mr. George Y. Wisner, the side of the flume at the lower weir was marked off into squares, which were lettered and numbered so that the line of the surface of the approaching and discharging water upon the side of the flume could be read probably within 0.05 to 0.10 ft. During all but the first two experiments these squares were read, and from these readings eight of the most characteristic profiles of the surface curves, shown in Figs. 21 and 22, have been plotted.

Comparing *L* and *M*, the effect of the rounded corner is readily seen in the reduction of contraction at the up-stream corner, particularly at the lower heads. The influence of the back slopes is seen in comparing *D*, *F*, *Q* and *S*. It is to be regretted that the readings were not continued up stream to the beginning of the surface curvature, which, in some cases, was lost in the rapid at the throat of the flume, 48 ft. from the crests. From some of the experiments upon the effect of contractions in pipes it seems very probable that this contraction may have seriously affected the discharge, and in future similar experiments it would seem well to remove it much farther from the weir or nullify its effect with baffles.

The weir *I* has a peculiar discharge curve. At low heads the flow is chiefly influenced by the 2 to 1 up-stream slope giving a high discharge, but as the head increases a point is reached where apparently the slope of the apron, 5 to 1, is not sufficient to maintain the velocity necessary to free the crest, and the discharge decreases relatively to that of a sharp-crested weir, the whole weir partaking, apparently, of the nature of a broad flat crest.

For the form of crest represented in Fig. 8, the Croton experiments, upon a large-sized model quite similar to this, indicate that the discharge partakes of the nature of that of the flat crest under high heads, the water between the crests of the old and new dam reducing the friction across the top, and thus producing or permitting a discharge slightly greater than that of a sharp edge, rather than giving one 20% less, as assumed by the author by comparison with the observation on crest *L*. This point can in no way be considered as a reflection upon the judgment of the author, as these data were not available at the time he made his estimates, and the matter is only presented to indi-

cate how far one may go astray on these questions, unless exact information has been obtained on the specific form considered. Mr. Williams.

One of the most important facts brought out in the past year's investigations in the Cornell Hydraulic Laboratory has been the formation of a vacuum more or less perfect behind the falling sheet when air is not freely admitted. With a weir 6 ft. high, the United States Deep Waterways Section, a head of 1.5 ft. has been observed to raise water behind the sheet to a height of 2 ft. above the level of the lower pool, and with a weir 8 ft. high and a 2-ft. head the water behind the sheet has stood 5 ft. above the level of the lower pool. The bottom boards of the plank aprons have been torn off frequently by the suction of the falling sheet at the toe of the dam. The possibility of a vacuum upon the down-stream face of a dam has, so far as the speaker is aware, never been considered in the design of such structures, but the pulling off of the granite facing on the down-stream side of the Austin Dam, while that at the crest remained practically intact, and other instances that have been reported of similar phenomena, seems to indicate that there was a very decided suction there on the occasion of its failure. This teaches that in the design of spillways, the practice of conforming them to the curve of the sheet, in order to obtain a smooth and compact overfall, should be reversed, and every precaution taken to prevent the sheet reaching the foot of the dam in a compact mass.

In conclusion, the speaker would acknowledge his great indebtedness for the very valuable services of his colleague, Mr. W. E. Mott, in the reduction and preparation of the data herein referred to and presented, and also in assisting in observing, under very trying conditions, during many of the later experiments.